CONDITION ASSESSMENT OF EXISTING BRIDGE STRUCTURES

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REPORT OF TASKS 2 AND 3 – BRIDGE TESTING PROGRAM

Prepared for

GEORGIA DEPARTMENT OF TRANSPORTATION

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ABSTRACT

Condition assessment and safety verification of existing bridges and decisions as to whether posting is required currently are addressed through analysis, load testing, or a combination of methods. Bridge rating through structural analysis is by far the most common procedure for rating existing bridges. Load testing may be indicated when analysis produces an unsatisfactory result or when the analysis cannot be completed due to lack of design documentation, information, or the presence of deterioration. The current rating process is described in the American Association of State Highway and Transportation Officials (AASHTO) *Manual for Bridge Evaluation, First Edition*, which allows ratings to be determined through allowable stress methods (AS), load factor methods (LF), or load and resistance factor methods (LRFR), the latter of which is keyed to the new AASHTO *LRFD Bridge Design Specifications*, which now is required for the design of new bridges, effective October, 2007. The State of Georgia currently utilizes the LF method. These three rating methods may lead to different rated capacities and posted limits for the same bridge, a situation that carries serious implications with regard to the safety of the public and the economic well-being of communities that may be affected by bridge postings or closures. To address this issue, the Georgia Institute of Technology has conducted a research program, sponsored by the Georgia Department of Transportation, leading to improvements to the process by which the condition of existing bridge structures in the State of Georgia is assessed and a set of *Recommended Guidelines for Condition Assessment and Evaluation of Existing Bridges in Georgia.* The research program has four tasks.

This report summarizes Task 2 – Bridge Diagnostic Load Testing Program and Task 3 - Bridge Evaluation by Advanced Analysis.

KEY WORDS:

Bridges; concrete (reinforced); concrete (pre-stressed); condition assessment; loads (forces); reliability; risk; structural engineering.

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EXECUTIVE SUMMARY

In the State of Georgia, approximately 9,000 bridges are identified in the state's Bridge Inventory Management System (BIMS) Database. This number is accurate as of October 2005, it includes pedestrian bridges, and rail road bridges that cross roadways; and excludes culverts. Most of these bridges have been rated by either Allowable Stress (AS) or Load Factor (LF) rating methods; approximately 2,000 of them have been found to require posting. The posting of a bridge results in economic losses that are related to the number of vehicles affected and the time required for detours that are necessitated by the load limits imposed by the posting. This study seeks to improve the current bridge evaluation techniques in the State of Georgia, as well as to contrast and critically appraise the three different AASHTO Rating methods that currently are permitted; Allowable Stress, Load Factor, and Load and Resistance Factor Rating (LRFR). The study objectives will be achieved through a coordinated program of diagnostic load testing and advanced structural modeling.

This study focused on three categories of bridges that make up 77% of the structures currently posted by the Georgia Department of Transportation (GDOT):

- Reinforced concrete T-beam bridges
- Steel girder bridges
- Pre-stressed concrete I-girder bridges

Older bridge structures that were designed for H-15 loading were targeted since they make up the largest design load group of posted structures.

This report documents the diagnostic test program conducted on four bridges in the State of Georgia between September, 2006 and May, 2007. These tests were designed to validate the finite element modeling process that is anticipated to play a central role in the development of improved bridge rating guidelines in the next phase of the study. The report explains why these four particular bridges were selected for in-depth examination, describes the bridge structural systems in detail, and summarizes the load ratings obtained using current AS, LF, and LRFR methods. The method of load testing, and finite element modeling are presented, and a preliminary comparison is made between the predicted and observed bridge response characteristics. Perhaps the most beneficial of these comparisons is one that demonstrates the increase in capacity of the reinforced concrete pier cap in shear that is gained by using the strut and tie model for the analysis of pier cap strength. The resulting combination of analytical and experimental results allows current rating procedures to be improved and provides a better understanding of the load-resisting mechanisms in common bridges in the State of Georgia.

CHAPTER 1 INTRODUCTION

1.1 BACKGROUND

Bridge structural systems in the United States are at risk from aging, leading to structural deterioration from aggressive environmental attack and other physical mechanisms, service demands from increasing traffic and heavier loads, and deferred maintenance. A condition assessment of an existing bridge may be conducted to develop a rating, confirm an existing load rating, increase a load rating for future traffic, or determine whether the bridge must be posted in the interest of public safety. In the State of Georgia, approximately 2,000 of its roughly 9,000 bridges have been determined to require posting by the Georgia Department of Transportation (GDOT) as of October 2005. Moreover, rating calculations have yet to be performed on approximately 1,600 of the bridges the GDOT monitors. Posting or other restrictive actions may have a severe economic impact on the State economy, which depends on the trucking industry for distribution of resources and manufactured goods.

Condition assessment and safety verification of existing bridges, and decisions as to whether or not to post them, can be addressed through analysis, load testing, or a combination of such methods. Bridge rating through structural analysis is the most common (and most economical) procedure for rating existing bridges, although load testing may be employed when analysis indicated a low rating or when the analysis cannot be completed due to lack of design documentation, or the presence of deterioration. The customary rating process is described in the American Association of State Highway and Transportation Officials (AASHTO)'s *Manual for Condition Evaluation of Bridges Second Edition (1994) with interim Revisions in 1995, 1996, 1998, and 2000*, which allows ratings to be determined through either allowable stress method (AS) or load factor method (LF). The State of Georgia normally has utilized the LF method for those bridges in the state that have been rated. A third rating procedure found in the *Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges* (2003),[1] is keyed to the new AASHTO Load and Resistance Factor Design (LRFD) method, defined in the *LRFD Bridge Design Specification, Fourth Edition* (2007). The LRFR method is being introduced to the bridge maintenance community, and some states are beginning to use it in developing their bridge ratings. These three competing rating methods may lead to different rated capacities and posted limits for the same bridge, a situation that cannot be justified from a professional engineering viewpoint and carries serious implications with regard to the safety of the public and the economic well-being of businesses and individuals who may be affected by bridge postings or closures.

[1] The *Manual for Condition Evaluation of Bridges* and the *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges* have been effectively combined in the new *Manual for Bridge Evaluation, First Edition* (2008), which became available in 2009 and permits rating by ASR, LFR or LRFR methods. A close scrutiny of the provisions in the new *MBE* has revealed that none of the findings and recommendations in the Report of Task 1are affected by the new document.

The Georgia Institute of Technology has been engaged in a multi-year research program, sponsored by the Georgia Department of Transportation, aimed at improving the process by which the condition of existing bridge structures in the State of Georgia is assessed. Improved guidelines for the evaluation of existing bridges will be developed by combining numerical structural analysis techniques with structural reliability. These guidelines will have a sound basis in structural engineering, allowing them to be updated as changing circumstances (traffic demands, additional data, material deterioration, etc) warrant, but will be presented in a relatively simply and familiar form that is suitable for implementation in routine rating assessments. A key ingredient of this research program is the validation of the results of the numerical structural analysis procedure by means of diagnostic load tests conducted on four bridges representing the type of structures that currently are of most concern to GDOT engineers. Upon successful validation of the numerical modeling approach, similar techniques can be used to extend the scope of the investigation to a broad selection of bridges, to conduct "virtual load tests" of that extended group, and to use those evaluations as a basis for critically appraising and revising, as appropriate, the current bridge rating process in the State of Georgia.

This report summarizes the work conducted in Tasks 2 and 3 of that research program, and documents the diagnostic test program conducted on four bridges in the State of Georgia between September, 2006 and May, 2007 for purposes of validating the finite element analysis process that is anticipated to play a central role in the development of improved bridge rating guidelines. We begin with a summary of the current inspection process, describe the criteria leading to the selection of bridges to be tested (in cooperation with GDOT engineers), describe the bridges and their instrumentation, describe the conduct of the load tests, and present some preliminary comparisons between computed and observed bridge response characteristics. The resulting combination of analytical and experimental results allows for the refinement of current rating procedures and provides a better understanding of the load-resisting mechanisms of common bridges in the State of Georgia. The report concludes with general recommendations for the planning of future load tests.

1.2 CURRENT AASHTO CONDITION ASSESSMENT MANUALS

All new ratings performed by GDOT utilize the LFR method as specified in the Second Edition of the *Manual of Condition Evaluation of Bridges* (AASHTO, 1994) with interim revisions through the year 2000. The ASR method also is used sometimes for prestressed girder bridges. This *Manual* is keyed to traditional bridge design methods, and accordingly contains provisions for rating with both the AS, and LF methods. However not all of GDOT's current bridge ratings have been determined by the LF method; 42% are rated based on the older AS rating method, and 6% of GDOT's bridges have yet to be rated by any method. Currently all 50 states are transitioning their new designs to the LRFD specification with the transition to be complete by October 2007 as per the recommendations of AASHTO and the FHWA (http://www.tfhrc.gov/pubrds/05jul/09.htm). Additionally there is an effort by many bridge experts to standardize all bridge ratings using the LRFR method. No date or mandate has yet been set as to when or if such standardization will occur (http://www.tfhrc.gov/pubrds/05jul/09.htm).

In order to evaluate the suitability of current and future rating practices it is necessary to understand the primary differences among the AS, LF, and LRFR rating methods. The FHWA requires all states to submit inspection results updated every two years through the Bridge Information Management System (BIMS) database on the structures in their jurisdiction. All three rating processes are designed around the information provided in this database.

1.2.1 Current Inspection Process

The current bridge rating procedures used by the Georgia Department of Transportation (GDOT) draw from a combination of bridge inspection reports performed every 2 years as per Federal Highway Administration (FHWA) regulations and from the available design documents. The rating analysis is based primarily on the design documents with reductions in capacity when damage or extreme degradation is observed by state bridge inspectors. When complete design documentation is not available; the basic component dimensions are measured by the inspector and are used in conjunction with typical material properties used in similar bridge structures during the time period of construction.

The deterioration of a bridge is quantified by its condition assessment number. This number is determined by a state inspector and is assigned separately for each major structural component type, including deck, superstructure (girders), and substructure (piers). The components are assigned a number on a scale from 0 to 9, as defined in Table 1.1. The LF and AS rating methods discussed in section 1.2 do not give explicit guidance as to how to reduce capacity for condition; however, the LRFR rating method defines a strength reduction factor of 1.0 for structural components receiving a condition assessment rating of 6 or better (Table 1.1), 0.9 for structural components with a condition assessment factor of 5, and 0.85 for structural components with a condition assessment factor of 4 or less. There is no other direct link between the rated capacity of a bridge and its condition assessment factors other than the specified reductions in capacity given by LRFR for bridges whose components hold low condition assessment values. The load carrying capacity of each structural component is determined separately and the condition assessment number attributed to a particular component, such as the substructure (piers), is used only in determining the capacity of that particular component.

Each bridge in the State inventory must be inspected every two years and the findings must be submitted to the FHWA as per FHWA guidelines. After the bridge's first inspection a rating analysis must be performed, relying on the condition assessment determined by the inspector and any other observations as to the physical state of the structure. With each subsequent inspection the previous inspection report is reviewed for any changes in the structure and appropriate adjustments are made to the rating analysis. Since the only link between the rating analysis and the condition assessment report is in the reduction of capacity at low condition ratings, a new rating analysis is only performed if significant deterioration of the bridge's structure is observed.

Table 1.1 Definition of Condition Assessment Scale from the FHWA Bridge Information Management System

Code	Description
N	NOT APPLICABLE
9	EXCELLENT CONDITION
8	VERY GOOD CONDITION – No problems noted
7	GOOD CONDITION – Some minor problems
6	SATISFACTORY CONDITION – Structural elements show some minor deterioration
5	FAIR CONDITION – All primary structural elements are sound but may have minor Section loss, cracking, spalling or scour
4	POOR CONDITION – Advanced section loss, deterioration, spalling or scour
3	SERIOUS CONDITION – Loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete.
2	CRITICAL CONDITION – Advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored, it may be necessary to close the bridge until corrective action is taken.
1	IMMINENT FAILURE CONDITION – Major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affection structure stability. Bridge is closed to traffic but corrective action may put back in light service.
0	FAILED CONDITION – Out of service. Beyond repair

1.2.2 Rating Approach

The Rating Factor (RF) of a bridge is the end result of all three rating methods. In each method it is computed using the same basic equation:

$$RF = \frac{C - (\gamma_{DC}) \cdot (DC) - (\gamma_{DW}) \cdot (DW) \pm (\gamma_p) \cdot (P)}{(\gamma_L) \cdot DF \cdot (LL + IM)} \tag{1.1}$$

$$C = \varphi_c \cdot \varphi_s \cdot \varphi \cdot R_n$$

Where:

RF = Rating Factor
C = Capacity
R_n = Nominal member resistance (as inspected)
DC = Dead-load effect due to structural components and attachments
DW = Dead-load effect due to wearing surface and utilities
P = Permanent loads other than dead loads
LL = Live-load effect
IM = Dynamic load allowance
DF = Distribution Factor
γ_{DC} = Load factor for structural components and attachments

γ_{DW} = Load factor for wearing surface and utilities
γ_p = Load factor for permanent loads other than dead loads
γ_L = Evaluation live-live load factor
φ_c = Condition factor
φ_s = System factor
φ = Resistance factor

The various load and resistance factors used in Equation (1.1) are used to account for uncertainties in both the strength of the structure and magnitude of the loads, and to provide a margin of safety to the rating process. In all three methods if the RF is greater than unity, the bridge is safe for the particular vehicle used as a live load for rating purposes; if it is less than unity, the bridge should be posted for that vehicle.

1.2.3 Comparison of Current Rating Methods

The primary differences among the AS, LF, and LRFR rating procedures are in their method of providing an appropriate margin of safety for the rating process, as defined by Equation (1.1). The AS method uses one resistance factor (or allowable stress factor) to obtain a margin of safety in its rating, while LF and LRFR use a combination of load factors and resistance factors. Each method requires that multiple categories of loading be considered, each with its own set of load and/or resistance factors. Table 1.2 shows some typical values for the various factors used in each method as well as the load categories associated with each factor. In rating by AS or LF, the Inventory rating is considered to be equivalent to the design of a new bridge, and utilizes the HS-20 load (Figure 1.1) and the safety factors used for the design of a new bridge designed by the AS or LF method. The Inventory rating analysis is performed first, and if the structure's RF determined by Equation (1.1) is larger than 1.0, then no further analysis needs to be performed. Otherwise, a rating is performed at the Operating level using the operating factors and the legal vehicles for the State (Figure 1.2); this operating rating is used to determine posting levels. However, current GDOT practice is to perform the actual structural analysis and rating of bridges by computer software so both levels of analysis are always performed regardless of whether AASHTO has deems them necessary.

In the LRFR method, the initial rating is termed a Design load rating and utilizes the HL-93 design vehicle stipulated in the AASHTO LRFD Specification (Figure 1.3) along with the load and resistance factors used in design of new bridges. If the design RF is greater than 1.0, further analysis need not be performed provided that the state legal loads fall within the envelope of the HL-93 Vehicle. If the RF is less than 1.0, an Operating load rating can be performed using the HL-93 vehicle and the operating factors. If this Operating load rating yields a RF>1 than the bridge is safe for AASHTO legal load (Figure 1.4) but not necessarily state legal loads. The Operating load rating is not mandatory; it can be skipped especially in states where the legal loads exceed the AASHTO legal loads, in which case a Legal load rating for that state is required regardless of the outcome of the Operating load rating (*LRFR Manual*, Sections 6.1.7.1 and 6.4.3).

Table 1.2 Values of Variables in Each Rating Method

ASD	**Inventory**	**Operating**
ϕ	Varies <1	
γ_L	1	1
γ_{DC}	1	1
IM*	<.3	<.3
DF**	>1	>1
LF	**Inventory**	**Operating**
ϕ	Material Dependent	
concrete	.9	
steel	1	
γ_L	2.17	1.3
γ_{DC}	1.3	1.3
IM*	<.3	<.3
DF**	>1	>1
LRFR	**Design**	**Legal**
ϕ	Material Dependent	
concrete	.9	
steel	1	
ϕ_c	1	
ϕ_s	1	
γ_L	1.75	1.35
γ_{DC}	1.25	1.25
γ_{DW}	1.25	1.25
γ_p	1	1
IM*	.33	.33
DF**	<1	<1

*The Impact factor I is dependent on the length of the span.
**The Distribution factor DF is dependent on the distance between girders.

1.2.4 Rating Vehicles

The AS and LF methods both use a series of legal loads (Figure 1.2) unique to each state and determined by its legislature for determining posting under the Operating rating category, as well as the generic HS-20 (Figure 1.1) loading that is used in the Inventory analysis. The HS-20 truck is a national rating vehicle and is used in conjunction with the Inventory analysis to provide a national standardized set of ratings for all Federal and State owned bridges, as mandated by the FMWA. In the HS-20 load, the concentrated loads represent the wheel loads along one wheel line and add up to half the total weight of the truck.

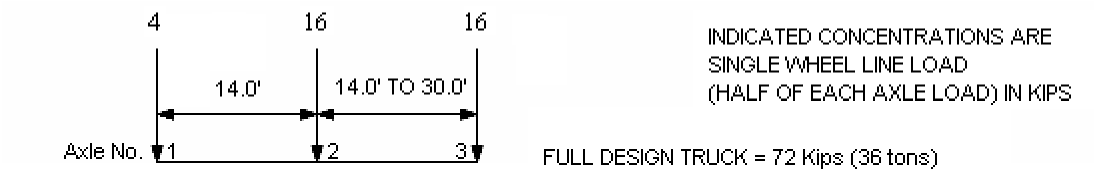

Figure 1.1 HS-20 rating vehicle
(Ref. AASHTO *Manual for Condition Assessment of Bridges* 1994)

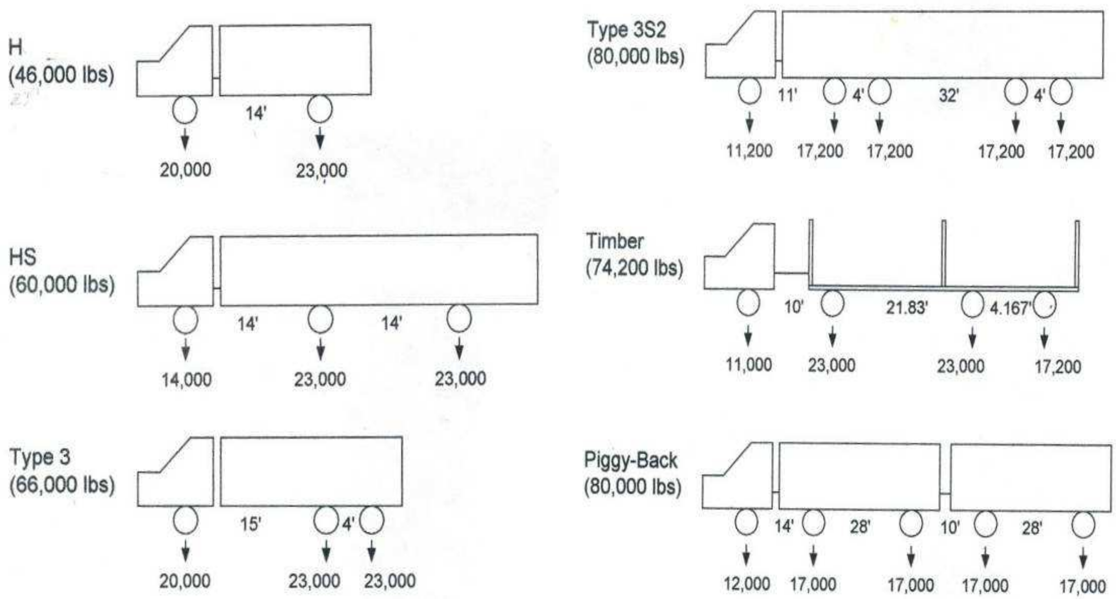

Figure 1.2 Georgia Legal Loads (Ref. GDOT)

Similar to the AS and LF rating methods, the LRFR method uses the legal loads shown in Figure 1.2, in addition to the HL-93 load (Figure 1.3) used in place of the HS-20 load for the design level rating. The HL-93 load is a combination of concentrated axle loads representing a truck and distributed loads representing automobile traffic. The HL-93 load also uses a set of different vehicle configurations instead of the one configuration represented by the HS-20 load. Note that in the LRFR method, the entire truck weight is used so the concentrated loads represent the load from each axle and sum to the entire truck's weight. This is significant because this practice is the primary reason that girder distribution factors in the LRFR method are less than 1, in contrast to the AS and LF methods where they are greater than 1.

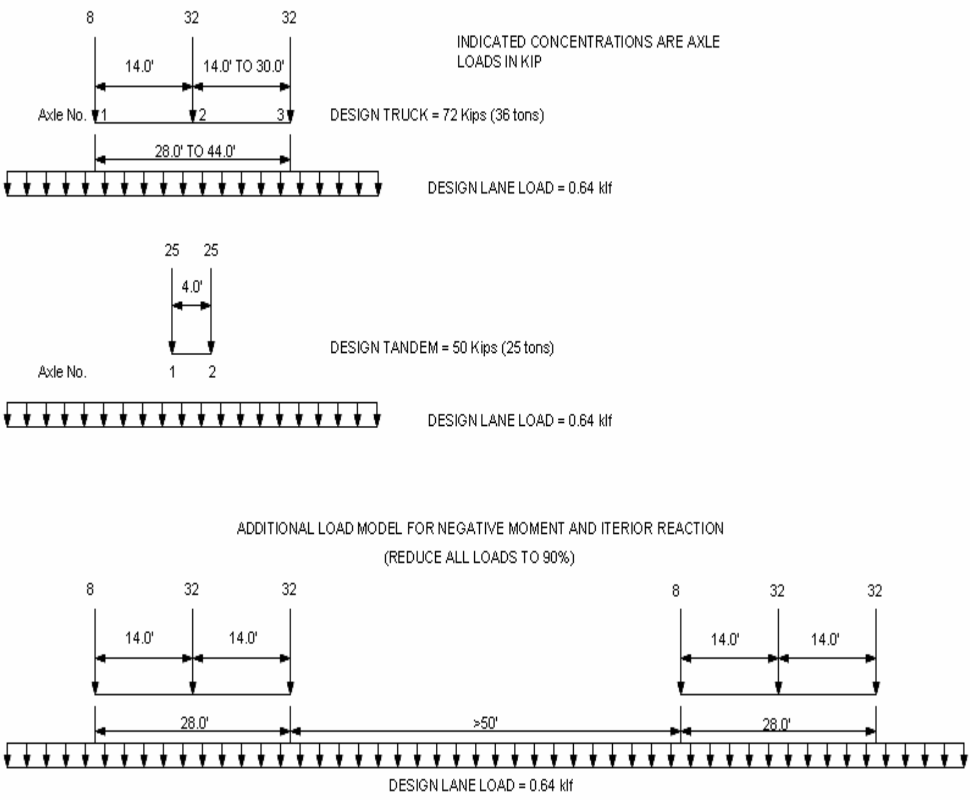

Figure 1.3 HL-93 Rating Vehicles (Ref. AASHTO *LRFR Manual* 2003)

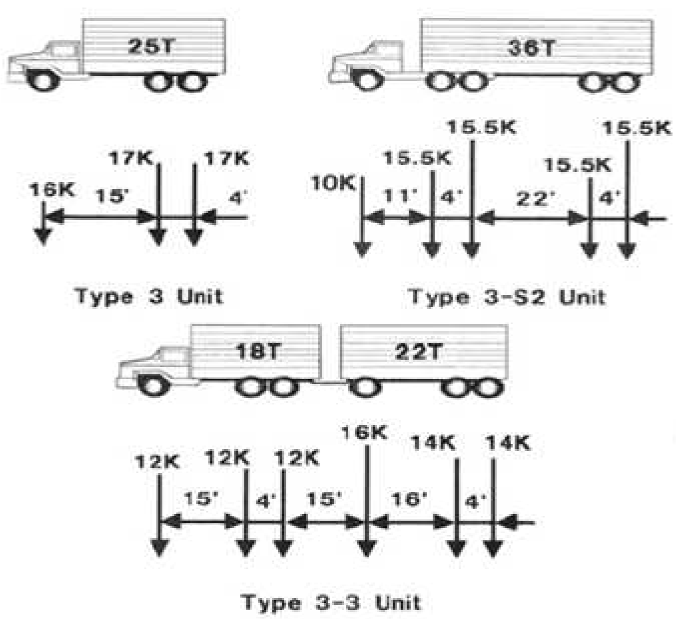

Figure 1.4 AASHTO Legal Loads (Ref. http://www.tfhrc.gov/pubrds/05jul/09.htm)

1.3 APPRAISAL OF AS, LF, AND LRFR RATINGS PROCEDURES

A survey was conducted of all state Departments of Transportation during the initial phase of the project.[2] Approximately 80% of state DOTs responded to the survey. Some responses to this survey indicated that the LRFR rating manual leads to significant (and unjustified) changes in posting values from the posting generated by AS and LF methods. One respondent remarked that, "For reinforced concrete bridges, the change from AS to LF resulted in approximately a 20% reduction in posting values. Changing from LF to LRFR will result in another 15% to 20% reduction in posting values for [reinforced] concrete. With LF and LRFR steel posting values increased." In contrast, the *LRFR Manual* section C6.1.7.1 states, "rating factors with HL-93 loads will generally be lower than previously calculated AASHTO ratings using the HS-20 loads for both Inventory and Operating levels." Such apparent differences in ratings and their underlying causes need to be explored further. Accordingly, an independent rating of each of the four bridges investigated in this study was conducted using the LFR, ASR and LRFR rating methods to identify sources of differences in the alternate rating procedures.

One major change in the rating methods between AS and LF is their method of estimating the structural capacity of the bridge. The AS method provides safety by calculating the capacity using a reduced ultimate capacity for the steel and/or concrete elements within the structure. AASHTO's 1994 *Manual of Condition Assessment* identifies each of these strength reductions factors separately for each structural material. In contrast, the LF method stipulates a single capacity reduction factor that depends on the failure mode (flexure, shear, compression) rather than the structural material, and uses load factors to increase the live and dead load to provide the desired margin of safety.

A second significant change between LF and LRFR ratings is the replacement of the HS-20 standard rating load with the HL-93 load consisting of a set of combined concentrated and distributed loads. However, the new HL-93 load is only used for the design of new bridges and for Inventory load ratings; thus, the final decision to post still utilizes the state Legal loads. Therefore, this change does not affect the posting analysis.

A third major change in the LRFR manual is the new load, resistance, and girder distribution factors that are keyed to the AASHTO *Manual for Load and Resistance Factor Design of Highway Bridges*. These ratings factors were keyed to the existing reliability levels of the AASHTO AS and LF methods.

The equations for nominal strength used in the LF and LRFR methods are, in most causes, identical. An exception is for shear in reinforced concrete, where LRFR introduces a new analysis technique termed compression field theory (Section 5.8.3.4.2 of the AASHTO *LRFD Design Specifications*) for use prestressed concrete members and deep beams where simple beam theory equations are not applicable.

[2] Wang, N., Ellingwood, B.R., Zureick, A.-H. and O'Malley, C. (2009). Condition assessment of existing bridge structures: Report of Task 1 – Appraisal of state-of-the-art of bridge condition assessment. GDOT Project RP05-01. (ftp://ftp.dot.state.ga.us/DOTFTP/Anonymous-Public/Research_Projects/)

CHAPTER 2 RESEARCH APPROACH

2.1 OBJECTIVES

In the State of Georgia, there are roughly 9,000 bridges identified in the state's 2005 Bridge Inventory Management System (BIMS) Database. Most of these bridges have been rated by either AS or LF rating methods and approximately 2,000 of them have been found to require posting (Figure 2.1). The posting of a bridge results in economic losses related to the number of vehicles affected and the time required for them to make necessary detours. Ratings of bridges on state or local routes instead of interstates are of particular interest for two reasons: first, these routes make up a much larger percentage of the state's bridges, and second, the repair or replacement of interstate bridges is typically (but not always) planned or conducted once the structure's Inventory load rating falls below 1.0, well before there is any need for posting. The severity of the economic impact of closure is what forces this early action for interstate bridges.

In order to accomplish the study objective of providing tools for improving bridge evaluation and rating techniques, a small subset of the state's bridges have been identified and extensively. This chapter explains the process of selecting these bridges.

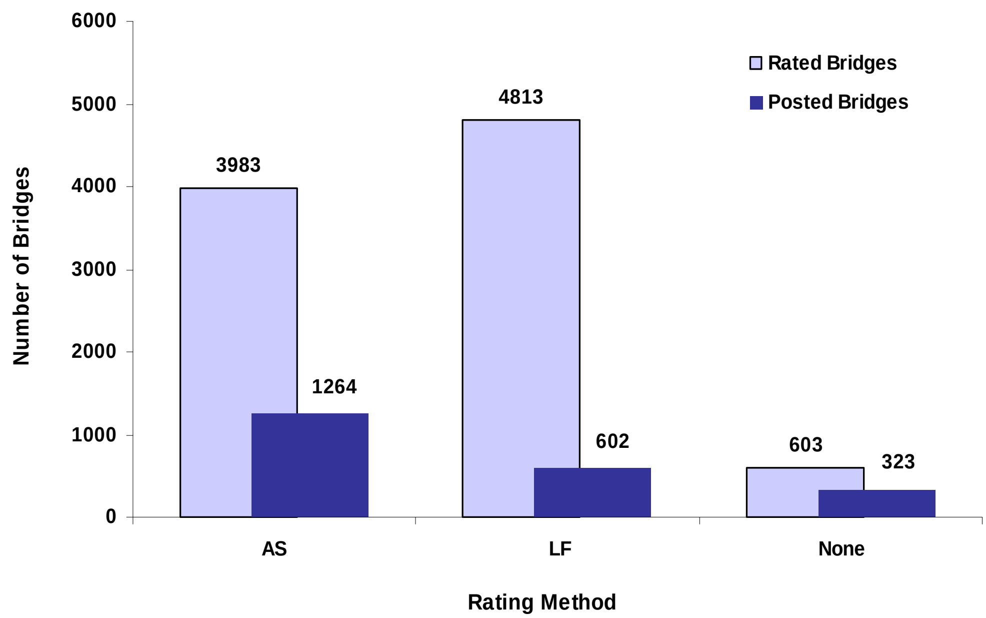

Figure 2.1 Rated and Posted Bridges, by Rating Method

2.2 BRIDGE SELECTION

In the State of Georgia, 82% of the bridges are multi beam/girder and Tee beam bridges. Discussions with the Georgia Department of Transportation (GDOT) engineers and evaluation of the status of the current bridge infrastructure in the State resulted in a list of several primary and secondary criteria for selecting the bridges to be involved in this testing program. This chapter reviews the criteria for selection and the process of identifying four bridges that represent the majority of the Georgia non-interstate bridge population and that will be analyzed and tested in later phases of the project.

When multiple bridges were available which meet all of the other selection criteria the one with the high average daily truck traffic (ADTT) levels was selected. Because small vehicle traffic represents a relatively insignificant load on a bridge, and bridges with high ADTT are more likely to have experience fatigue and other time-dependent strength degradation effects due to their large number of load cycles. Railroad bridges and dedicated pedestrian bridges were not considered because they are outside the jurisdiction of the Georgia DOT. The State bridge inventory was scanned and, based on the criteria identified below; approximately 15 bridges were identified and visited by the project team.

2.2.1 Design Load

The HS-20 load is currently used by GDOT in bridge rating and was used for design of new bridges until the adoption of LRFD and the HL-93 design load. Georgia like all others states has until October 2007 to transition to designing all of its new bridge structures by the LRFD specifications. However many of the older bridge structures that require rating were designed for loads smaller than HS-20. The HS-15, H-15, and H-20 loads (Figure 2.2) are some of the design loads used prior to the adoption of the HS-20. In the past different design loads were utilized on rural versus urban roadways; for example, to cut costs in rural areas the HS-15 or H20 load could be used to design a bridge instead of the HS-20 load; however as legal loads have increased over time these older design loads have been gradually phased out of use. Bridges designed for the HS-20 load are fairly new, and are unlikely to have experienced significant deterioration or loss of strength. Thus, this project will focus on bridges designed for H-15 loads. In Figure 2.3 it is clearly that the bridges designed for the H-15 load represent not only the largest number of any single design category but also the largest number of posted structures where the initial design strength is known.

2.2.2 Structure Type

Of the bridges that the GDOT has posted, 77% fall into one of three categories:

- Reinforced concrete T-beam bridges, representing 21%;
- Deteriorated steel girder bridges, representing 53%; and
- Pre-stressed concrete I-girder bridges, representing 3%.

While the posted pre-stressed bridges represent a much smaller number than the steel or reinforced concrete girder bridges, they are of particular concern because a high number of relatively new pre-stressed bridges are posted. It is unclear whether the posting of these pre-stressed bridges is due to the inadequate capacity of the prestressed girders, the pier caps, or the deck. However, the GDOT expressed concern over this bridge category, and it was included in this study for this reason. Of the pre-stressed girder bridges that are posted, 57% were constructed after 1980; in contrast, 2% of posted reinforced concrete bridges were constructed after 1980, and 10% of posted steel bridges were constructed after 1980. Figure 2.4 shows the primary structure type of bridges constructed over each decade from the 1940's to present. Figure 2.5 identifies the number of bridges from each category that have been posted as unfit for some or all of the state legal load vehicles (Figure 1.2).

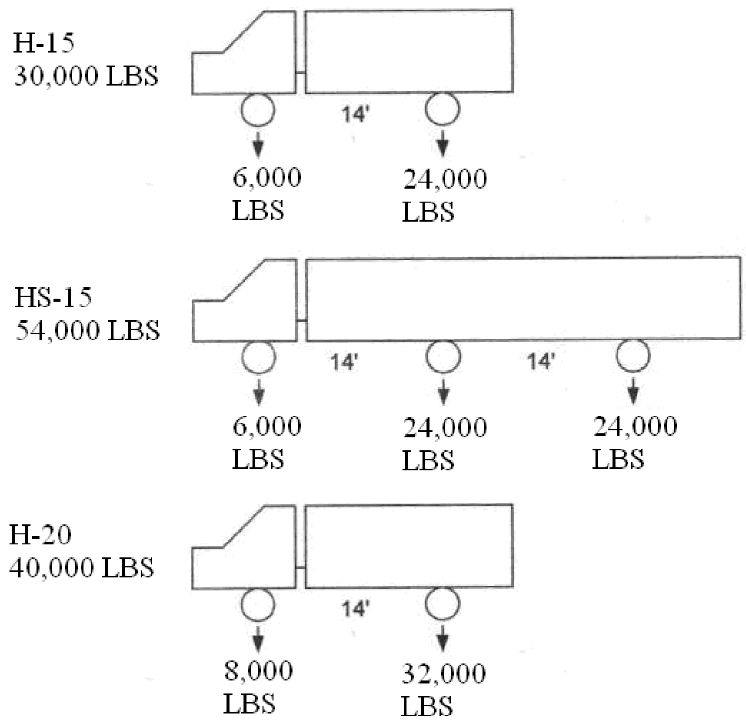

Figure 2.2 Design Loads Used Prior to HS-20 Load

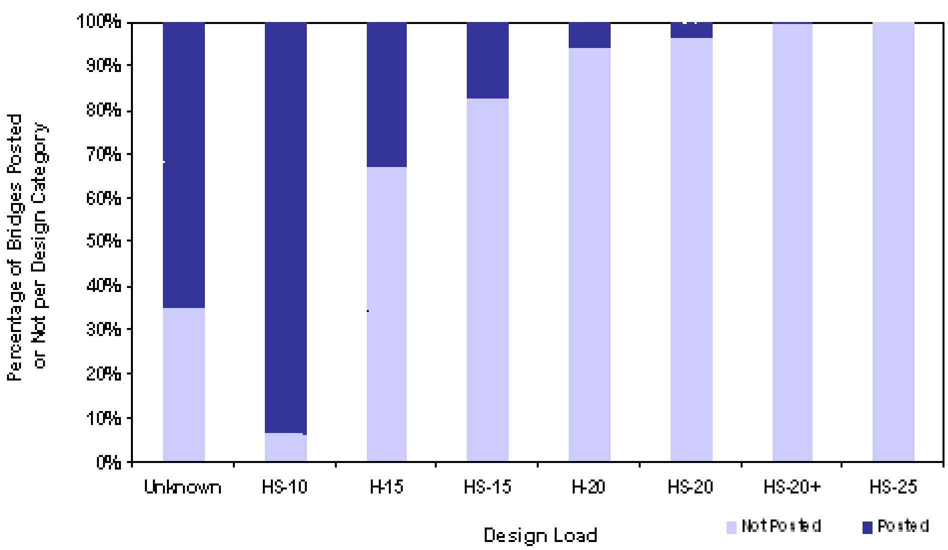

Figure 2.3 Posted Bridges by Design Load

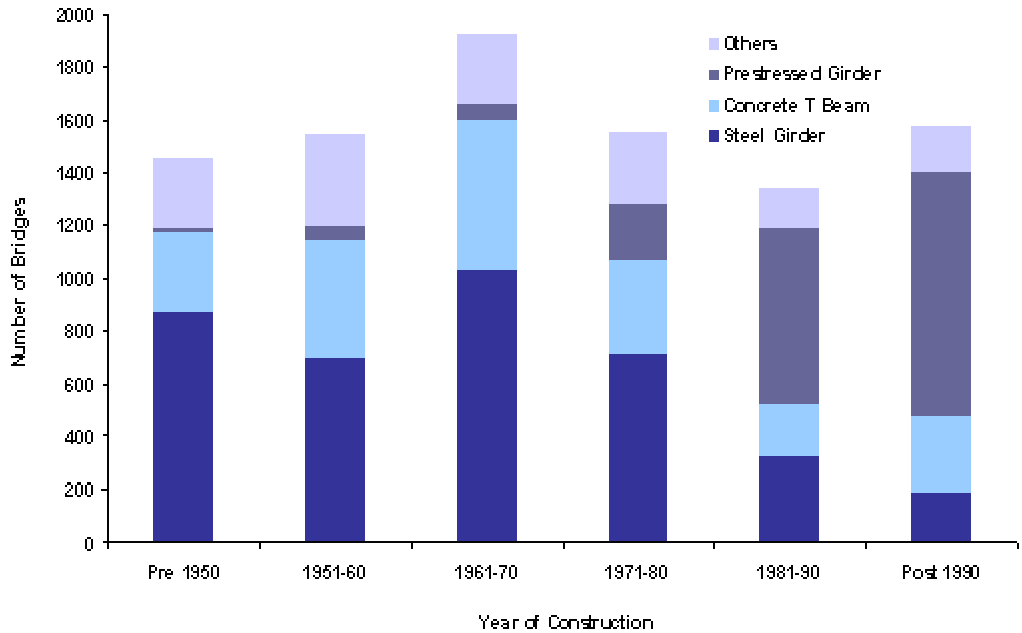

Figure 2.4 Bridge Categories Identified by Decade of Construction

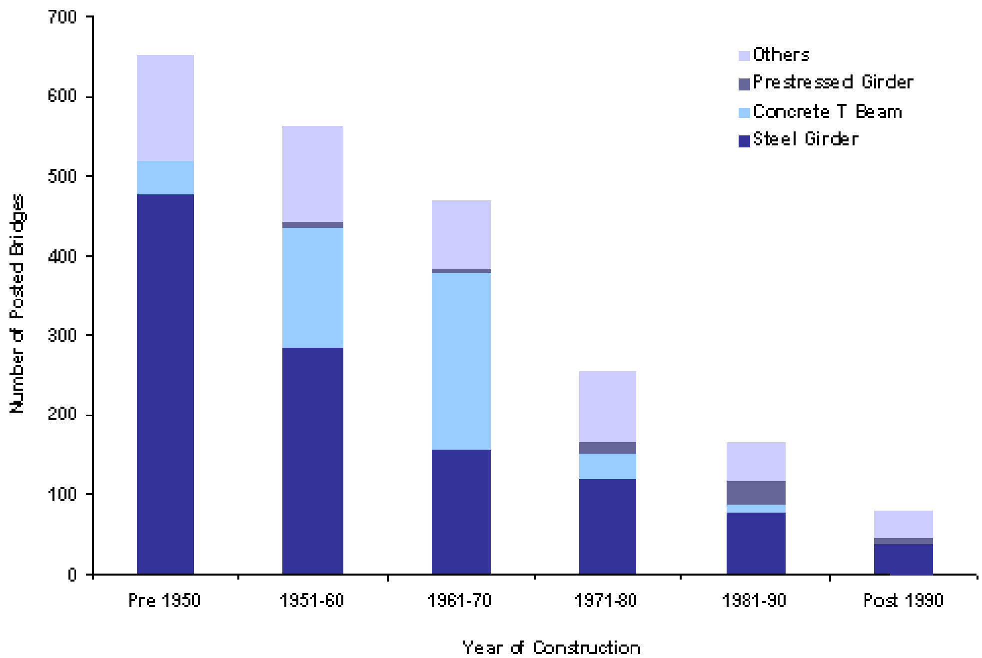

Figure 2.5 Posted Bridges Identified by Decade of Construction

Wooden bridges have been omitted from this study because they represent less than 2% of the bridges in the State and are typically historic structures intended only for light automobile traffic. Suspension, truss, and other long span bridges also represent a small percentage of the total population and often require independent rating procedures; accordingly, they are not addressed in this study.

2.2.3 Bridge Condition

The condition rating of a bridge (discussed in section 1.2) represents the GDOT bridge inspector's assessment of the overall condition of each of the structures primary components. For this study, bridges with moderate condition ratings, typically 5-7, were selected. Highly deteriorated structures such as those with condition assessment levels below 4 were not chosen because, by definition, they suffer from significant deterioration that needs to be addressed in the near future. As such, these highly deteriorated structures would need to be posted or repaired and are unlikely to benefit from a refined rating procedure. At the opposite end of the scale, bridges in very good condition were not chosen due to the fact that these typically are new structures designed to modern load levels and unlikely to require posting. However, whatever evaluation procedure is developed for moderately deteriorated bridges designed to old standards would also be applicable to bridges in good condition that were designed to those same standards.

2.2.4 Accessibility and Ease of Instrumentation

The screening of the GDOT's bridge inventory based on structure type, age and design load led to a bridge population that was still far too large to conduct an in-depth analysis of each structure. Thus, a series of secondary criteria was employed to narrow the selection to a manageable number of bridges. The first of these was all bridges that spanned interstates, railroads or very large rivers were eliminated due to the inaccessibility of their superstructure or substructure for field instrumentation without special equipment. Second, all bridges that had been widened or otherwise modified by adding different types of girders were eliminated from consideration; many of these bridges were T-beam bridges that had been widened by the addition of pre-stressed girders. Pre-stressed concrete box girders were also eliminated as these represent a small portion of the state's bridges, and are beyond the scope of this investigation. Finally, candidate bridges for analysis and diagnostic testing were limited to those within approximately 50 miles of Atlanta (and each other) in order to provide greater efficiency in the load testing and inspection process.

2.3 FINAL BRIDGE SELECTION

Site visits were conducted for the 15 bridges identified by the above criteria. During the inspection of these bridges, many factors not easily discernable from or present in the bridge database were observed and used in the final selection. Some of these observations included flange local buckling in steel H-piles, local terrain features that might impact ease of instrumentation and testing, and concerns about the validity of assuming composite or non-composite action in girder bridges. On this basis, four bridges were selected for detailed analysis and diagnostic load testing. These four bridges are discussed in detail in Chapters 3 through 6 of this report.

- Bridge #1 - Straight T-Beam Bridge State ID# 129-0045-0 (Gordon County),
- Bridge #2 – Skew T-Beam Bridge State ID# 015-0108-0 (Bartow County),
- Bridge #3 – Pre-Stress Bridge State ID# 223-0034-0 (Paulding County), and
- Bridge #4 – Steel Girder Bridge State ID# 085-0018-0 (Dawson County)

Each chapter outlines the manual-based rating, bridge instrumentation and load testing, and presents a comparison of the load test results with the results of a finite element analysis of each bridge.

CHAPTER 3 FIELD TESTING OF A REINFORCED CONCRETE T BEAM BRIDGE WITH STRAIGHT APPROACH

3.1 DESCRIPTION

3.1.1 Bridge Details

This bridge is a reinforced concrete T-Beam structure, which is located in Gordon County GA, and carries State Route 156 over Oothkalooga Creek approximately 1 mile west of Calhoun, GA. It was designed according to the AASHTO 1953 *Design Specification* for H-15 traffic load, constructed in 1957, and is not posted. The centerline of the bridge is unskewed with respect to the girder supports. Figure 3.1 shows an overview of the girder and bridge structure, while Figure 3.2 presents a schematic labeling each of the bridge's spans and bents.

Figure 3.1 Bridge 129-0045-0 Overview

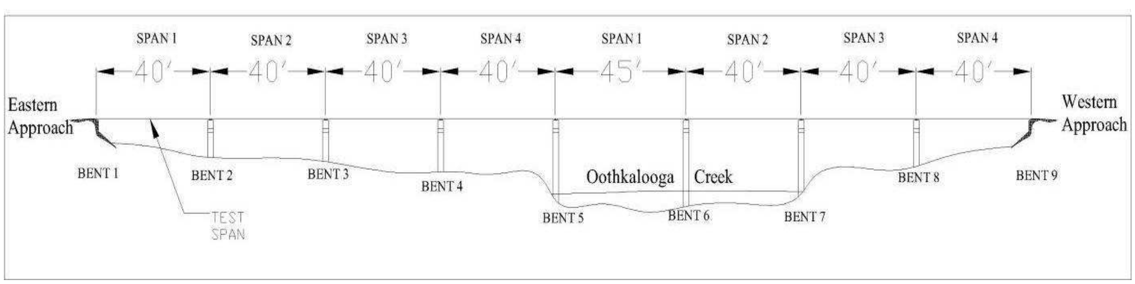

Figure 3.2 Elevation of Bridge 129-0045-0

As shown in Figure 3.2, the bridge is 325 ft (99.1 m) long with eight simple spans, seven of which are 40 ft (12.2 m) in length from center line to center line of expansion joints and one of which is 45 ft (13.7 m). The most accessible span (and the one chosen for instrumentation and load-testing) is the easternmost span, which is only 10 ft (3.3 m) above the ground.

The bridge has a reinforced concrete deck that is 32.3 ft (9.8 m) wide and a roadway width of 25.7 ft (7.8 m) as shown in Figure 3.3. Girders in spans 1, 2, 3, 4, 6, 7, and 8 are 24 in (609.6 mm) deep, and span 5's girders are 31.75 in (806.5 mm) deep. All of the girders are 18 in (457.2 mm) wide and spaced 7.3 ft (2.2 m) on center. All of the girders and the 6 in (152.4 mm) deep slab were cast monolithically. The design documents indicate that the girders, slab, and piers are all constructed of 2,500 psi (17.2 MPa) concrete reinforced with 40 ksi (276 MPa) steel reinforcing bar. Figure 3.4 illustrates the general distribution of reinforcement in the composite girders and slab. Details such as bar lengths and bends are contained in the design drawings held by the GDOT. Both the GDOT load rating performed in accordance with the LF rating method and the independent rating performed in this study determined the girder shear capacity to be the limiting factor in this bridge's rating.

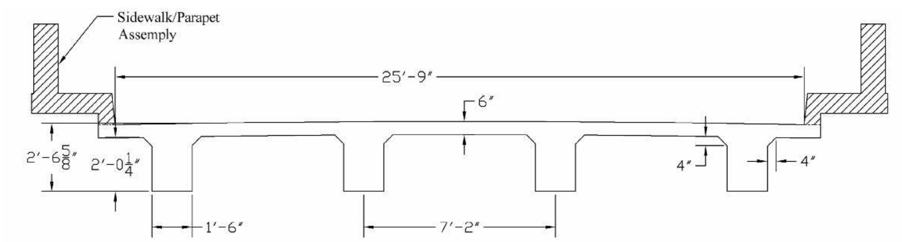

Figure 3.3 Cross Section of Bridge Deck and Girders (Ref. GDOT Drawing)

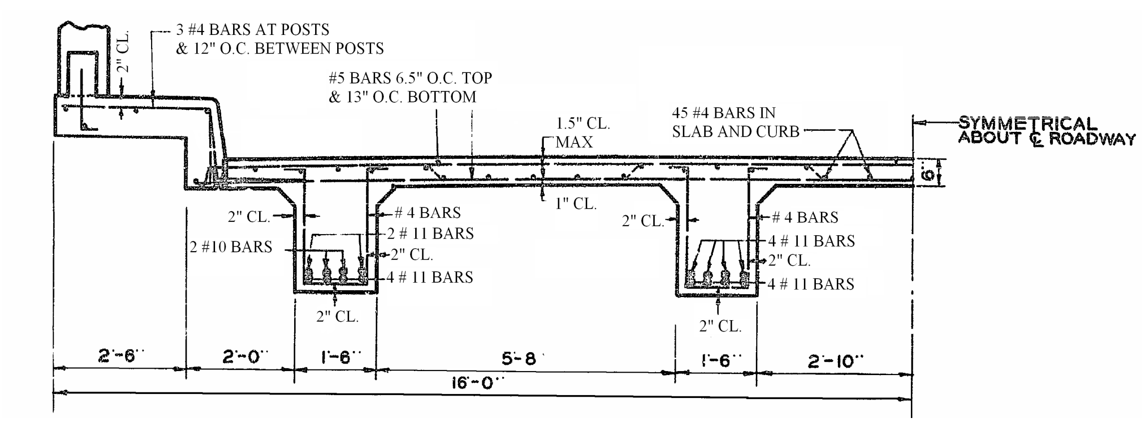

Figure 3.4 Deck and Girders Reinforcement Detail (Ref. GDOT Drawing)

The two end spans are supported by reinforced concrete abutments. Supporting the girders between spans are hammerhead bents, depicted in Figure 3.5. Each bent consists of a single, square reinforced concrete column under the centerline of the bridge and a tapered beam extending out in both directions to support the four girders. The depth of the cantilever beam varies from 4 ft 4 in (1321 mm) at the pier support to 2 ft 6 in (762 mm) at the tip. The focus of the field load testing is on the superstructure (girders) because, as noted previously and discussed in detail in Section 3.1.3, the rating calculations indicated that the girders are the weakest components of the bridge.

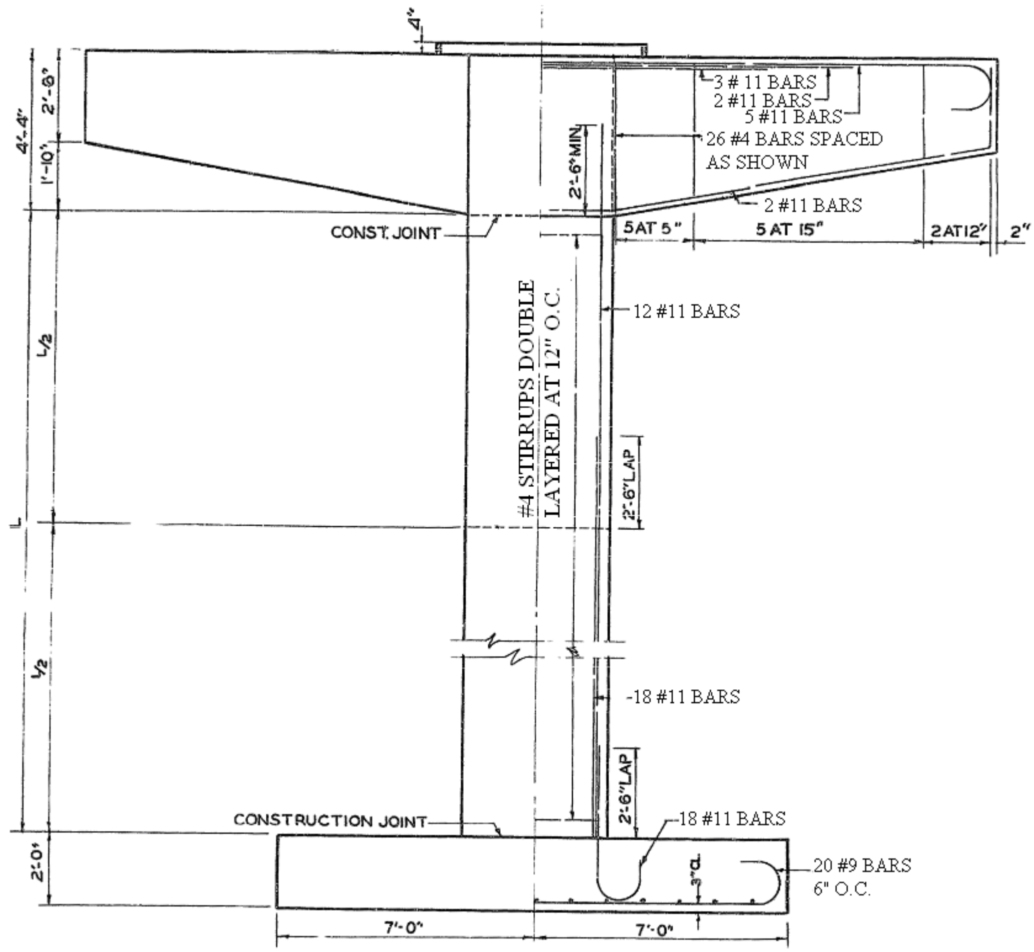

Figure 3.5 Pier Cap, Column, and Footing Detail (Ref. GDOT Drawing)

3.1.2 Bridge Condition

According to the GDOT bridge inspector's report, the deck has undergone moderate surface deterioration, scaling, and cracking and has been patched multiple times. The inspector did not identify the location or size of specific cracks or patches; however, upon visiting the structure, the patches were easily distinguishable from the underside of the bridge. In most cases, plywood still covered the underside of the patch, and where the plywood had been removed, the outline of each patch was visible. The bridge deck was given a condition assessment rating of 5 (cf Table 1.1) by the GDOT inspector. The supporting reinforced concrete girders were less deteriorated and the inspector gave them a condition rating of 7. Upon initial inspection, while preparing the girders for instrumentation, only minor hairline cracking in the girders was noted, as reported in the GDOT inspection report. However, after grinding and cleaning the girder surfaces for the purpose of instrumentation, flexural cracking at mid-span was clearly visible (Figure 3.6).

Figure 3.6 Flexural Crack in Girder

According to the GDOT inspection report, the concrete bents and piers that comprise the substructure of the bridge showed hairline cracks and several areas of exposed cap reinforcement, but during instrumentation no exposed reinforcement was found in the test span. The GDOT inspector judged that none of these deficiencies were a significant problem and assigned the substructure a condition rating of 6. No exposed reinforcement in the substructure components supporting the test span was observed during instrumentation.

3.2 RATING PROCEDURE

All condition ratings for this bridge were determined at the time of the latest GDOT inspection, dated June 15, 2005. The GDOT engineer performed the LF based posting analysis using the state legal loads and the HS-20 load. This analysis of the structure showed the governing structural component to be shear in the superstructure and that it was capable of supporting a 21 ton H-type truck, 25.8 ton HS-type truck, 24 ton Tandem truck, 39.6 ton 3-S-2 truck, 31.8 ton Logging truck, or a 52.8 ton Piggy-back truck. Based on these load capacities and the Georgia Legal Loads in Figure 1.2 this structure should be posted for the H, HS, Tandem, 3-S-2, and Logging Vehicles. However, the GDOT bridge management records indicated no need for posting, and no posting signs were found at the bridge site.

The GDOT rating results for the superstructure were found to be in good agreement with the LFR-based rating performed independently by the research team in this study using the HS-20 loading case. Table 3.1 shows the comparison of the state's rating and the LFR ratings performed independently.[3] In light of the comments received from the Task 1 survey, ASR and LRFR ratings also were performed to assess the effects that changing rating methods would have on the rated capacity of this particular bridge.

Table 3.1 Bridge rating results in Tons for HS-20 vehicle

	Inventory		Operating	
Rating Method	Flexure	Shear	Flexure	Shear
ASR (Current Study)	25.2	14.8	45.0	27.0
LFR(GDOT Calculation)	-	**15.5**	-	**25.8**
LFR (Current Study)	27.0	**15.5**	45.0	**25.9**
	Design		Legal	
LRFR (Current Study)	Not Comparable		33.5	22.0

Note that the LRFR method uses the HL-93 load for the Design ratings instead of the HS-20 load so it has been omitted from Table 3.1. The Operating Load level is used in both LFR and ASR ratings for the Inventory rating category, while the Legal Load level is used in the LRFR rating to determine bridge postings. For bridge 129-0045-0, all three rating methods found it incapable of carrying the 36 Ton HS-20 Truck and in need of posting. However, the posting value of the governing component (shear in the girders) are 4% lower using LFR rather than ASR rating, and would be 23% lower using the LRFR method rather than the ASR method, as shown in Table 3.1. This difference is in the range of values identified for reinforced concrete structures in the Task 1 survey discussed in Section 1.3 of this report. As discussed in Chapter 1, the primary change among the three methods affecting posting is in the modified load, resistance, and distribution factors found in Table 1.1.

3.3 FINITE ELEMENT ANALYSIS

To obtain further insight into the behavior of bridge 129-0045-0, a finite element (FE) analysis was conducted. The finite element modeling and analysis were performed using the commercially available software package ABAQUS (Ref. ABAQUS, 2006). A detailed description of the analysis can be found in Appendix A of this report. The FE analysis was developed using data from construction documents provided by the GDOT.

3.4 INSTRUMENTATION PLAN

3.4.1 Instrumentation Location

The diagnostic load test was designed to validate the FE analysis of the bridge. In order to assist in the design of the load test, FE analyses were conducted prior to testing.

[3] Details of the rating calculations for this bridge are found in Appendix C of the Task 1 report.

In these analyses the load consisted of trucks that approximate the actual test trucks. The results of these preliminary analyses were then used to determine the placement of the instrumentation and as a basis for monitoring the results during the test.

It was decided to compare the global performance of the bridge to that of the FE analysis through measuring the deflection of the girders. Accordingly, the instrumentation of the bridge consisted of both potentiometers and mechanical dial gauges, used for redundancy, (Figure 3.7 and 3.8). The gauges were all supported by stable elevated platforms and a cable was hung from the bottom of each girder down to the platform below using hooks epoxied into place (Figure 3.9).

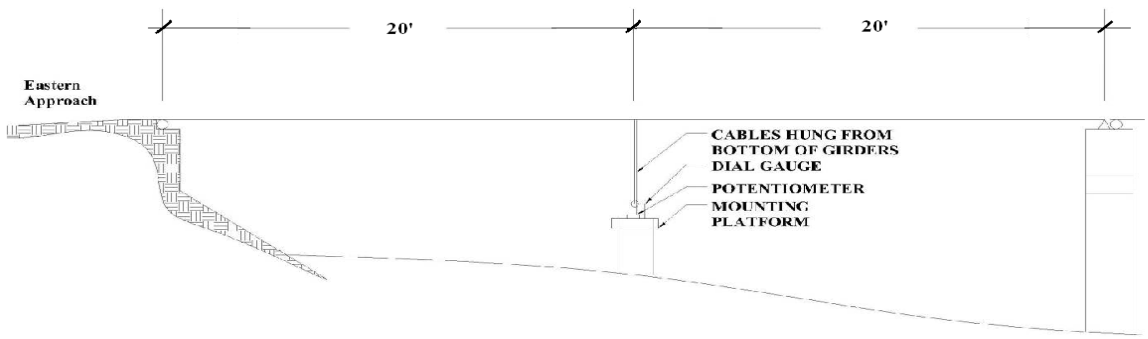

Figure 3.7 Instrumentation Location – Elevation

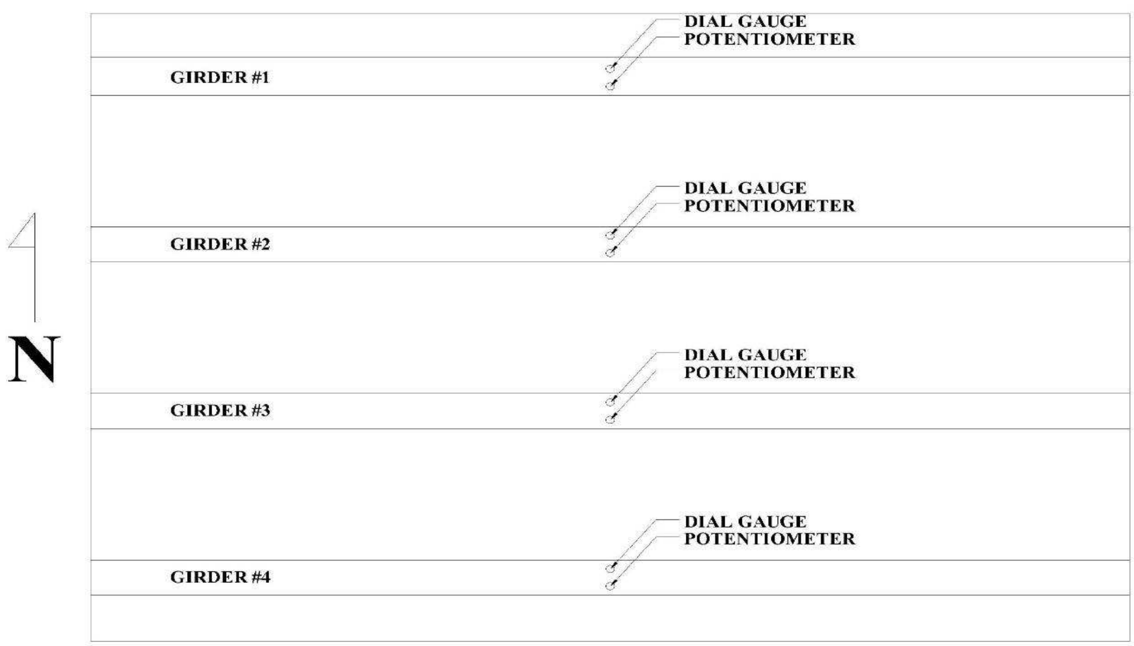

Figure 3.8 Instrumentation Location – Plan View

Figure 3.9 Mounting Base (Left), Cable Connecting Gauges to Girders (Right)

The dial gauge setup is shown in Figure 3.9 and 3.10. An extension cable was added to the potentiometers and dial gauges to account for the height of the span (Figure 3.9) over the platform.

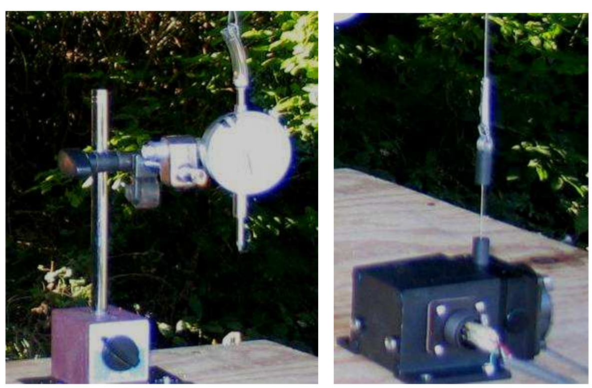

Figure 3.10 Dial Gauge (left), Potentiometer (right)

3.4.2 Calibration of Instrumentation

Each potentiometer and dial gauge used in the test was calibrated in the Structural Engineering and Materials Laboratory of the School of Civil and Environmental Engineering, Georgia Institute of Technology using an Extensometer Calibrator made by Russell Gauges LTD, as shown in Figure 3.11.

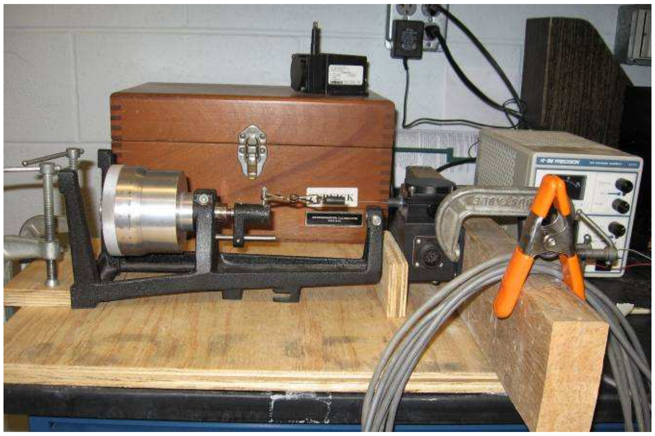

Figure 3.11 Laboratory Calibration of Gauges

The calibration was performed by stepping the extensometer in ¼ in increments from 0 to 1 in and back to 0, and then repeating this in ½ in increments. A reading was recorded at each increment and then the results plotted verses the expected values (Figure 3.12). In the case of the potentiometer shows below the max difference between the reading of the potentiometer and the extensometer was 0.009 in, and the average difference was 0.001 in. For the four potentiometers used in this test the maximum deviation from the extensometer measured in the laboratory was 0.028 in.

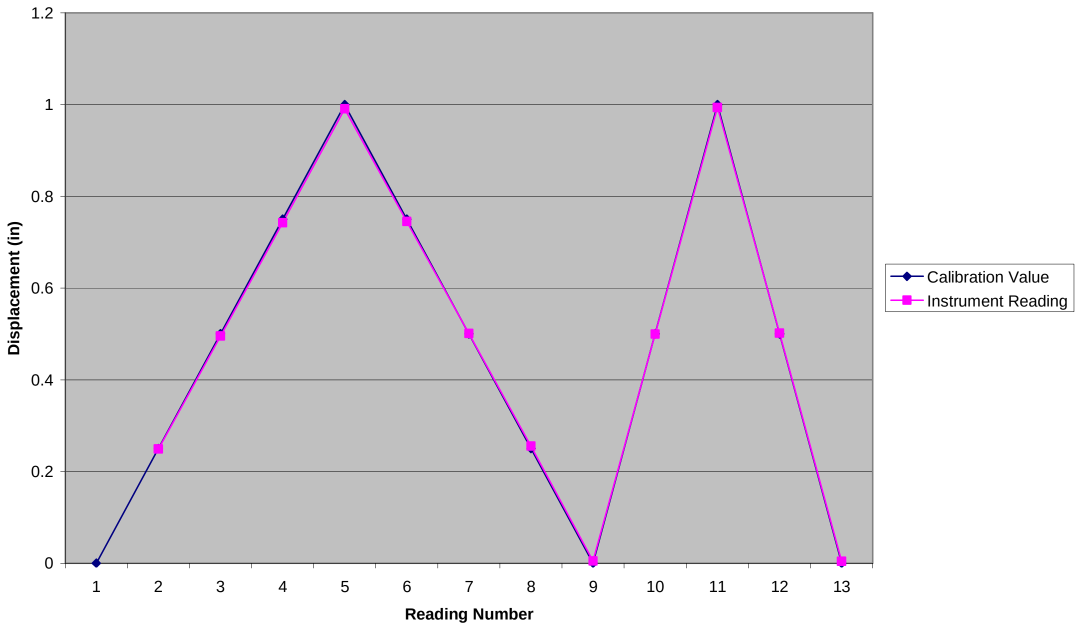

Figure 3.12 Potentiometer Calibration Test Plot

The potentiometers were monitored using a DAC-pro data acquisition system supplied by Omega Engineering Inc. The DAC-pro system has a total of eight data channels and was configured to take readings for this test on manual command. Once the readings were taken and recorded, the trucks were moved off the test span. A 400-Watt XPower PowerSource Portable Inverter supplied by Xantrex Technologies provided AC power. The AC power was converted to a 10 volt DC input signal by a BK Precision DC Power Supply for the Celesco PT1DC Cable Extension Transducers (potentiometers).

3.5 DIAGNOSTIC LOAD TEST RESULTS

3.5.1 Testing

The bridge load test was conducted on September 26, 2006 on the easternmost span (Figure 3.7), with the gauges mounted at mid-span as shown in Figure 3.8. The load on the bridge consisted of 4 GDOT tandem axle dump trucks (Figure 1.2 Type 3 truck) that were weighed individually before arriving at the bridge site. Table 3.3 summarizes the axle and total weights of the trucks used in the load test.

Table 3.2 Truck Weight (lb) Details for Oothkalooga Creek Bridge Test

	Load on Axle 1	Load on Axle 2	Load on Axle 3	Overall Truck Weight
TRUCK 1	18,400	19,100	19,000	56,500
TRUCK 2	19,100	17,400	17,100	53,600
TRUCK 3	19,500	19,300	19,000	57,800
TRUCK 4	17,800	18,700	18,600	55,100

The bridge was closed during each of three loading repetitions and reopened to traffic between repetitions to limit traffic congestion. Each repetition began and ended by recording the reading from each gauge with no load on the test span; subsequently, the four trucks were backed onto the middle of the instrumented span, one truck at a time, starting with truck #1 and ending with #4 (Figure 3.13).

Data were then recorded from the DAC-pro instrumentation system and the mechanical dial gauges. Finally the trucks were moved off the test span and traffic was permitted to resume. Figure 3.14 shows the stages of the loading process.

3.5.2 Analysis and Bridge Load Test Results

During the testing the gauges appeared to be functioning properly. Spot checks of the measured deflections generally agreed with the results of the FE analysis which was performed prior to the test with expected truck weights. The cracks noted previously in the girders showed no visible signs of opening or lengthening during the load testing of the bridge structure.

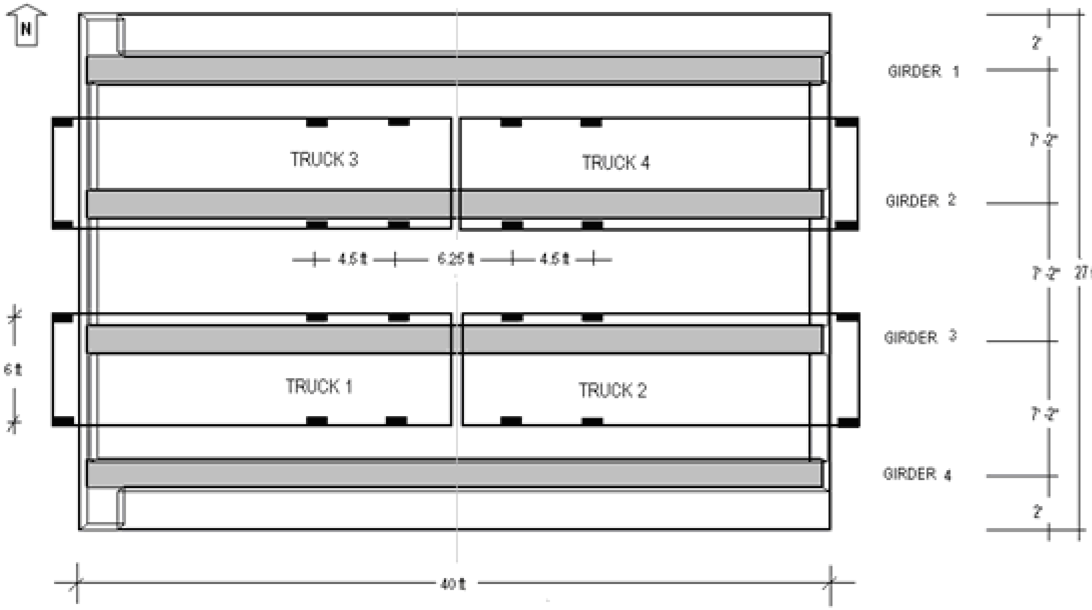

Figure 3.13 Placements of Trucks for Gordon County Bridge Test

Figure 3.14 1 truck (top left), 2 trucks (top right), 3 trucks (bottom left), 4 trucks (bottom right)

Following the test, the recorded girder deflections were analyzed and compared to those of the FE analysis. The dial gauge results were judged to be unreliable because of the mounting technique; these results are not presented, but the mounting was corrected for the other bridge tests. The deflections measured by the potentiometers were found to be in very good agreement with the FE analysis results.

Three loading cases are presented here: two, three, and four trucks on the bridge span. Figure 3.13 shows the truck locations and girder numbers for each of the three cases. In the first load case, trucks 1 and 2 were positioned on the same side of the bridge, as shown in Figure 3.13; the results are presented in Table 3.3 and Figure 3.15. The second load case used trucks 1 through 3; the results are compared to the FE analysis results in Figure 3.16 and in Table 3.4. The final load case utilized all four trucks, leading to the results in Figure 3.17 and Table 3.5.

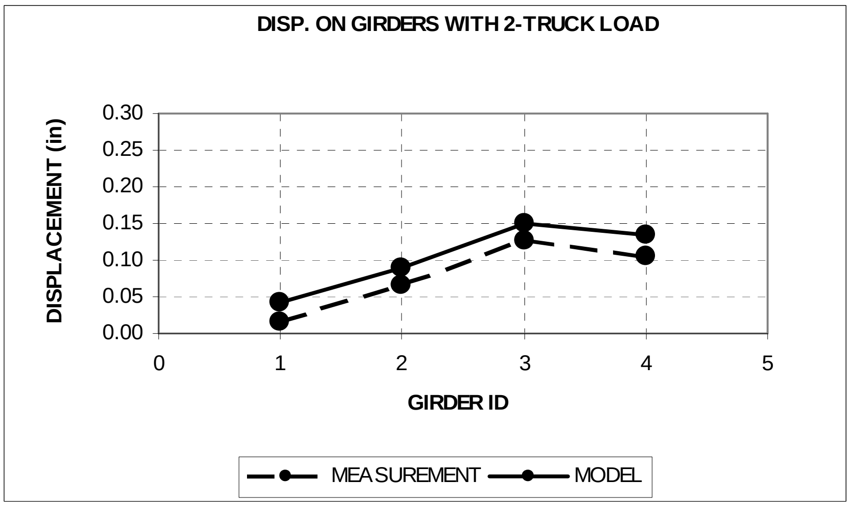

Figure 3.15 Girder Displacements Under 2 Trucks

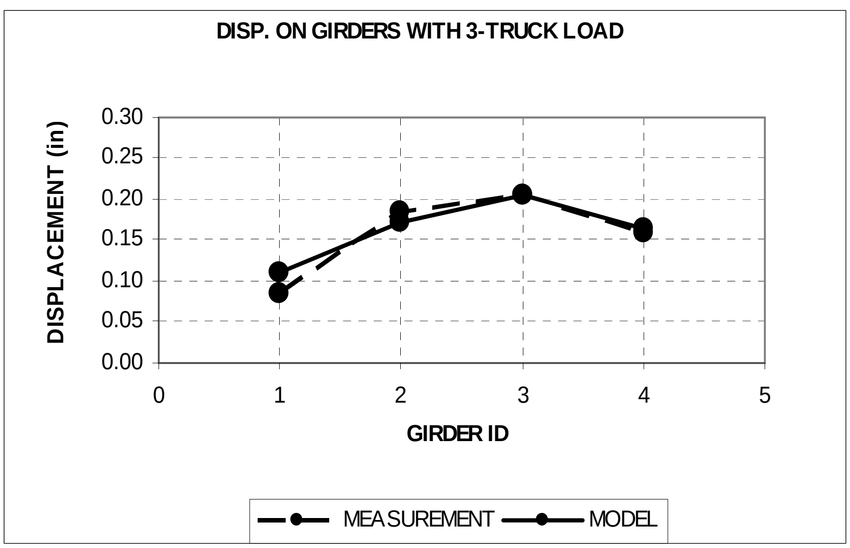

Figure 3.16 Girder Displacements Under 3 Trucks

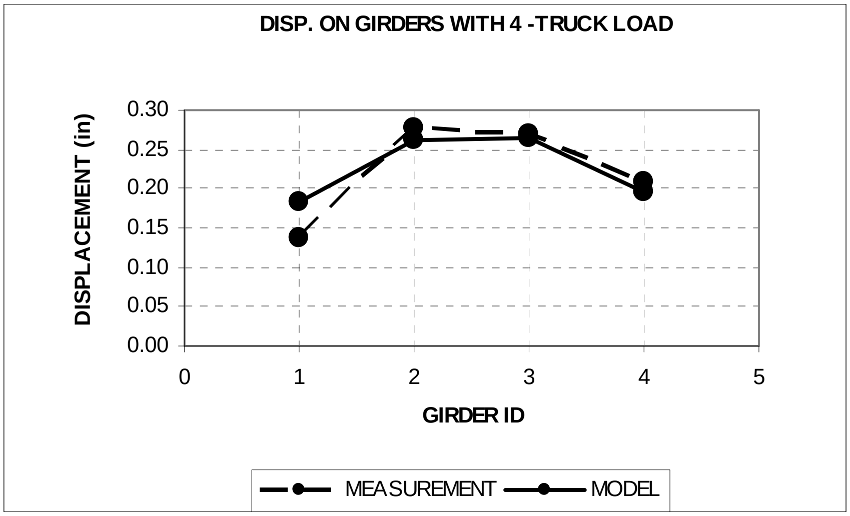

Figure 3.17 Girder Displacements Under 4 Trucks

Comparisons of the results of the physical load tests and finite element analyses are presented in Figures 3.15 - 3.17 and Tables 3.3 - 3.5. These results indicate that a properly constructed FE analysis captures the load-deflection relationship for the individual reinforced concrete girders and the load distributing pattern among the girders accurately.

Table 3.3 Test and FE Analysis Deflections – 2 Trucks

2 Trucks	**Dial Gauges**			
Test #	G1	G2	G3	G4
1	0.016	0.066	0.126	0.105
2	0.014	0.064	0.125	0.104
Measurement	**0.015**	**0.065**	**0.125**	**0.105**
FE Analysis	**0.0415**	**0.0886**	**0.1487**	**0.1338**

Table 3.4 Test and FE Analysis Deflections – 3 Trucks

3 Trucks	**Dial Gauges**			
Test #	G1	G2	G3	G4
1	0.080	0.187	0.209	0.165
2	0.085	0.185	0.203	0.159
3	0.087	0.185	0.205	0.156
Measurement	**0.084**	**0.185**	**0.205**	**0.160**
FE Analysis	**0.1092**	**0.1706**	**0.2041**	**0.1629**

Table 3.5 Test and FE Analysis Deflections – 4 Trucks

4 Trucks	Dial Gauges			
Test #	G1	G2	G3	G4
1	0.139	0.283	0.273	0.206
2	0.145	0.272	0.269	0.208
3	0.127	0.273	0.269	0.210
Measurement	**0.137**	**0.276**	**0.270**	**0.208**
FE Analysis	**0.1825**	**0.2618**	**0.2654**	**0.1947**

3.5.3 Discussion and Conclusions

In the three and four truck loading cases, the maximum difference between the girder deflections calculated by the FE analysis and those measured during the test was 25% for Girder 1, and less than 10% for girders 2, 3, and 4. Considering the fact that the *in situ* strength (and thus stiffness) of the concrete and strength of steel in the bridge are unknown, this agreement is excellent. The difference between calculated and measured deflections of the girders in the two truck loading case was larger, but the girder distribution patterns were similar. Much of this difference can be attributed to the fact that the deflections under two trucks are much smaller than under four trucks, but the margin of error of the sensors does not decrease. With the FE analysis results in good agreement with the measured results, it is concluded that the FE model represents this particular bridge's overall performance accurately. In terms of the overall performance of the bridge, the load imposed by the four trucks was approximately 1.85 times larger than the H-15 truck for which the bridge was designed; nevertheless, its maximum measured deflection was 0.276 in (7.01 mm), far less than the 0.6 in (15 mm) deflection corresponding to the limit on deflection of span/800 stipulated by AASHTO's LRFD bridge specification.

The FE analysis of this bridge was based on concrete and steel strengths provided by the GDOT construction documents. Additional core samples were requested from GDOT for this bridge because it was scheduled to be demolished and replaced. These samples were obtained in May, 2008 at the time the bridge was demolished. The analysis of the *in situ* compression strength from testing these samples is described in detail in the report of Task 4.[4]

[4] Ellingwood, B.R., Zureick, A.-H., Wang, N. and O'Malley, C. (2009). "Condition assessment of existing bridge structures: Report of Task 4, Part I – Development of guidelines for condition assessment, evaluation and rating of bridges in Georgia." Report of Project GDOT No. RP05-01, Georgia Department of Transportation, Atlanta, GA (ftp://ftp.dot.state.ga.us/DOTFTP/Anonymous-Public/Research_Projects/).

CHAPTER 4 FIELD TESTING OF A CONCRETE T-BEAM BRIDGE WITH A SKEW APPROACH

4.1 DESCRIPTION

4.1.1 Bridge Details

Bridge 015-0108-0 is located in Bartow County GA, and carries Old Alabama road over Pumpkinvine Creek approximately 3.7 miles west of Cartersville, GA. Figure 4.1 shows an overview of the bridge and its girder and slab structure.

Figure 4.1 Bridge 015-0108-0 Overview

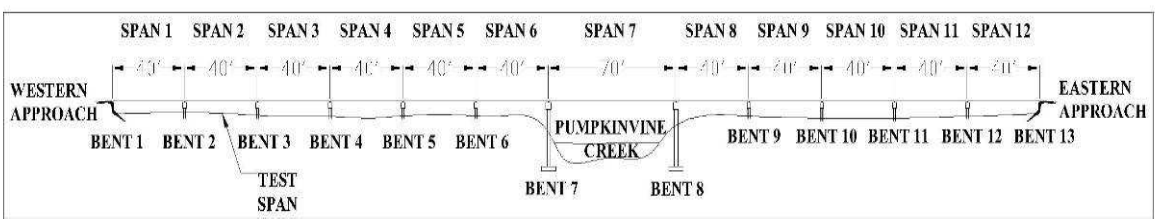

Figure 4.2 Bridge 015-0108-0 Elevation

The bridge was designed using the 1977 AASHTO specification for HS-20 loading and was constructed in 1979. It is 510 ft (155 m) long overall, and is comprised of eleven 40 ft (12.2 m) reinforced concrete T-beam simply supported spans and one 70 ft (21 m) prestressed I-girder span (Figure 4.2). The bridge was initially selected in part because GDOT's BIMS database reported it as being designed for H-15 load but it was later discovered that the bridge was designed for HS-20 loading. The decision was made to continue with field testing this bridge because it was posted, despite its apparent good condition upon visual inspection and the fact it had been designed to current design loads.

The prestressed concrete span crosses over Pumpkinvine Creek, while the other spans all comprised of reinforced concrete beams cross five to ten feet (1.52 – 3.05 m) over the flood plain. The centerline of the bridge is skewed at an angle of 60 degrees with respect to the girder supports. The bridge has a deck width of 40.25 ft (12.3 m) and a roadway width of 40 ft (12.2 m). The 40 ft (12.2 m) spans are comprised of five 25.25 in (641.35 mm) deep by 18 in (457.2 mm) wide girders that are cast monolithically with the 7.75 in (196.85 mm) thick slab (Figure 4.3).

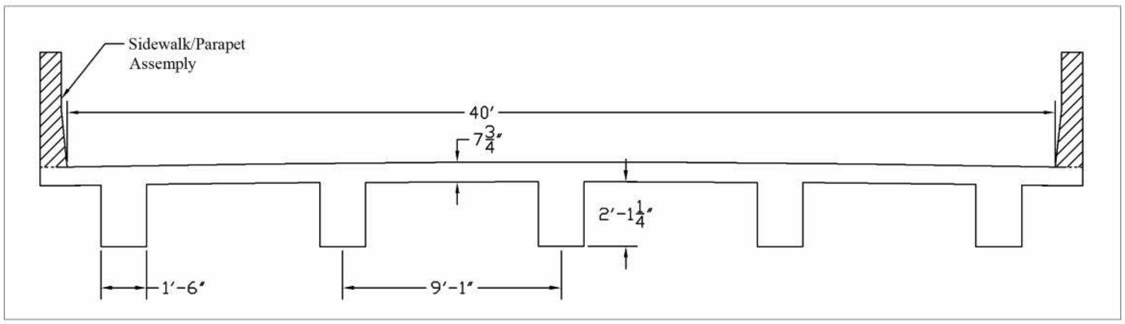

Figure 4.3 Cross Section of Bridge Deck and Supporting Girders (Ref. GDOT Drawings)

The girders, pier caps, and slab are all constructed of 2,500 psi (17 MPa) concrete reinforced with 40 ksi (276 MPa) steel reinforcing bars. Figure 4.4 shows a cross section of the reinforcement in the girders and slab.

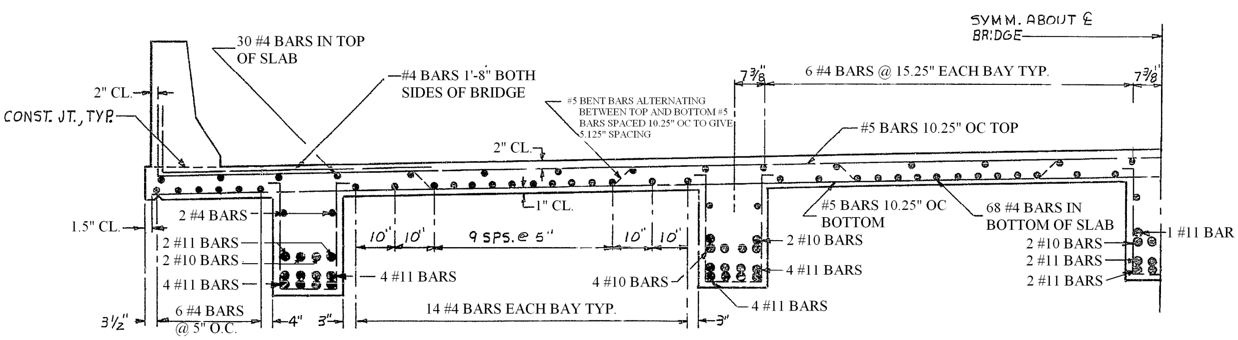

Figure 4.4 Deck and Girder Cross Section, Showing Reinforcement Details (Ref. GDOT Drawings)

Reinforced concrete pier caps on steel H piles support the girders at bents 2-6 and 9-12 (numbered in Figure 4.2), as illustrated in Figure 4.5. Each bent consists of a 30 in (762 mm) wide by 24 in (609.6 mm) deep reinforced concrete pier cap, supported at the location of all five girders by steel H 12x53 piles. Each pile is located directly below a girder; as a result, the reinforced concrete bent transfers very little shear between piles and most of the load from each girder is applied to the pile directly below that girder.

Bents 7 and 8 are comprised of a pier cap supported by two columns and a footing all constructed of reinforced concrete. Finally, bents 1 and 13 are reinforced concrete abutment founded on H-piles.

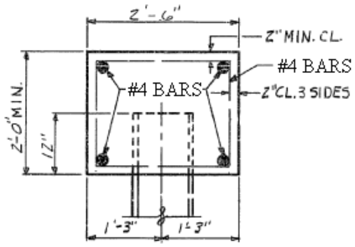

Figure 4.5 Pier Cap Cross Section A-A (Ref. GDOT Drawings)

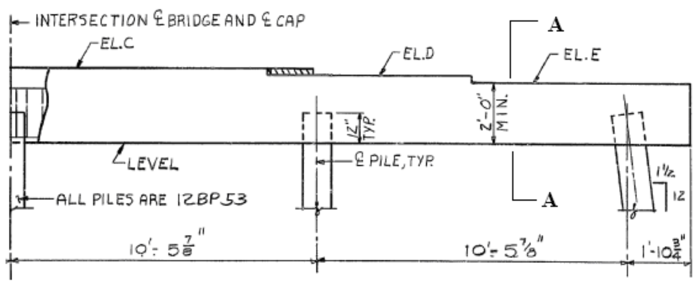

Figure 4.6 Pier Cap Elevation (Ref. GDOT Drawings)

4.1.2 Bridge Condition

According to the GDOT inspector's report, July 11, 2006, the deck has a condition assessment rating of 7. The GDOT inspector reported minor cracking and spalling in the supporting reinforced concrete T-beam girders at their supports, and gave them a condition rating of 6. When the girder surface was ground prior to attaching instrumentation for the load test both flexural cracks (Figure 4.7) at mid-span and shear cracks at the supports of the span became clearly visible.

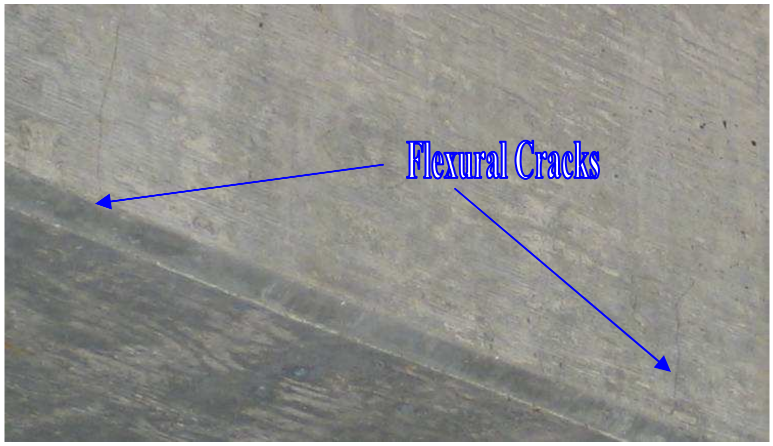

Figure 4.7 Flexural Cracks in Girders Found After Cleaning Surface

The GDOT inspector found minor cracking and spalling in a number of the bents and abutments, but judged that none of them required repair. He also reported that the steel piles of bent 6 along one of the creek embankments needed cleaning, painting, concrete encasement, and sway bracing, and that the embankment needed to be reinforced with rip-rap to prevent continued scour. During instrumentation, the concrete pier caps showed no significant deterioration, but the piles showed sign of flange local buckling (Figure 4.8) approximately at mid-height between the ground and pier cap in the affected piles. The most prominent buckling was found in the piles along the creek bank (bent 6). The piles along the creek banks have unsupported lengths as long as 15 ft (4.6 m) while the piles over the flood plain have unsupported lengths of only 5 ft (1.52 m). All of the piles are H12x53 piles and their buckling load capacity, taken from AISC 13th Ed., is 488 kips (2171 kN), based on an unbraced length of 5 ft (1.52 m), and $b_f/2 * t_f$ equal to 13.8. There are two likely sources for the observed damage: either the bridge was subjected to an excessively overweight vehicle at some time during its service life or the damage was caused during pile driving and not noted in the bridges file. The flange local buckling was not noted by the state inspector, who assigned the substructure a condition rating of 6.

Figure 4.8 Evidence of Flange Local Buckling

4.2 RATING PROCEDURE

All of the condition ratings were determined at the time of the latest inspection using the LF method. The GDOT engineer's calculations determined that the bridge required posting. The resulting postings were governed by the capacity of the substructure and determined the bridge to be able to carry H20-mod type truck to a maximum weight of 21.5 tons, tandem truck to 19.4 tons, logging truck to 24.7 tons, HS20-mod truck to 22.1 tons, 3S2 truck to 31.2 tons, and Piggy back to 40 tons.

This bridge was also rated in this study using the LFR, ASR and LRFR rating methods, evaluating the superstructure under the HS-20 loading case.[1] Table 4.1 compares the State's rating results to the present LF, AS, and LRFR based results.

Table 4.1 Bridge Rating Results in Tons for HS-20 Vehicle

	Inventory		Operating	
Rating Method	Flexure	Shear	Flexure	Shear
AS (Current Study)	49.0	33.8	78.1	51.8
LF (GDOT Calculations)		**29.0**		**48.4**
LF (Current Study)	46.8	**30.2**	78.5	**50.4**
	Design		Legal	
LRFR (Current Study)	Not Comparable		63.7	37.8

Note that the HL-93 load is used with the Design ratings in LRFR rather than the HS-20 load so it has been omitted from Table 3.1. The LRFR Legal rating level and for the AS and LF Operating rating level are used to determine the bridge's posting load. The LRFR method indicated a 19% decrease in the rated capacity for flexure and a 25% decrease in the rated capacity for shear as compared to the ASR. This again is in general agreement with the results of the Task 1 survey discussed in Section 1.3.

4.3 FINITE ELEMENT ANALYSIS

Finite element analyses of the bridge were conducted prior to field testing, with loading that approximated the anticipated test trucks used in the field test. The finite element models provided further insight into the performance of the bridge under various truck loading conditions. The results of the analyses also were used to determine the placement of the instrumentation and as a basis for comparison of the results during the test. The same analysis approach as used in Section 3.3 and detailed in Appendix A was employed in the FE analysis.

4.4 INSTRUMENTATION PLAN

The FE analysis was validated by measuring the deflections of each of the girders at the locations identified in Figures 4.9 and 4.10. The potentiometers were supported by

[1] Details of this rating are provided in Appendix D of the Report of Task 1.

a stable platform and a cable was hung from the bottom of the girders down to the platform (Figure 4.11).

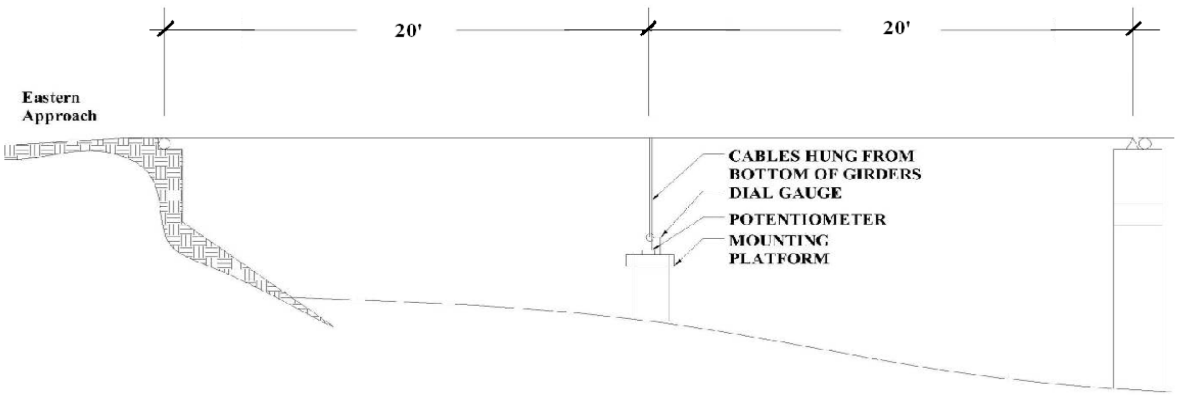

Figure 4.9 Elevation of Instrumentation Location (not to scale)

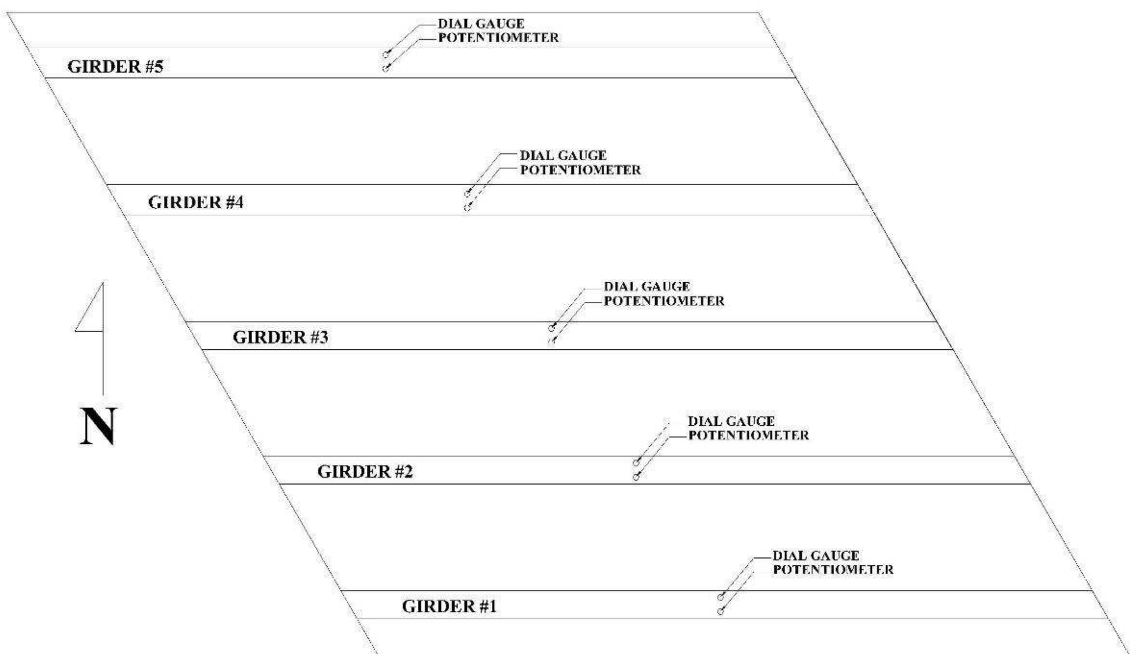

Figure 4.10 Instrumentation Location (not to scale)

During this test, one dial gage was used at each of the five girders, and was placed on blocks so that its probe was pressed directly to the bottom of the girder. This prevented the malfunction that was observed in the first bridge test. Figure 4.11 illustrates the setup for such instrumentation. Prior to field testing all deflection measuring devices were calibrated in the Structural Engineering and Materials Laboratory by the method presented in Section 3.4.2.

Figure 4.11 Gauge Mounting Chapter 3 Test (left), Chapter 4 Test (right)

4.5 DIAGNOSTIC LOAD TEST RESULTS

4.5.1 Testing

The load test was conducted on September 28, 2006 on span number 2 (Figure 4.2). Deflection-measuring devices were placed at mid-span. The loading on the bridge consisted of four GDOT tandem axle dump trucks (Figure 1.2: Type 3 truck) that were weighed individually before arriving at the bridge site. The truck and axle weights of the test vehicles are summarized in Table 4.2.

Table 4.2 Truck Weight (lb) details for Pumpkinvine Creek Bridge Test

	Load on Axle 1	Load on Axle 2	Load on Axle 3	Overall Truck Weight
TRUCK 1	18,300	17,700	16,700	52,700
TRUCK 2	18,500	18,500	18,400	55,400
TRUCK 3	17,300	19,000	19,000	55,300
TRUCK 4	19,000	18,800	18,800	56,600

Two load cases were employed during this load test. In the first, trucks 1 and 2 were placed in the same lane, in the locations noted in Figure 4.11. The second load case involved all four trucks positioned as shown in Figure 4.11, representing 2 full lanes of traffic. Several repetitions of each test were performed to establish reproducibility, with the bridge closed during each of the loading repetitions and reopened to traffic between repetitions to prevent traffic congestion. Each repetition began and ended with recording the reading from each gauge with no load on the test span; subsequently, the four trucks were backed onto the middle of the instrumented span, one truck at a time, starting with truck #1 and ending with #4 (Figure 4.12). Deflection measurements were then recorded from the DAC-pro instrumentation system and the mechanical dial gauges. Finally the trucks were moved off the test span and traffic was permitted to resume. Figure 4.13 illustrates two stages of the loading process.

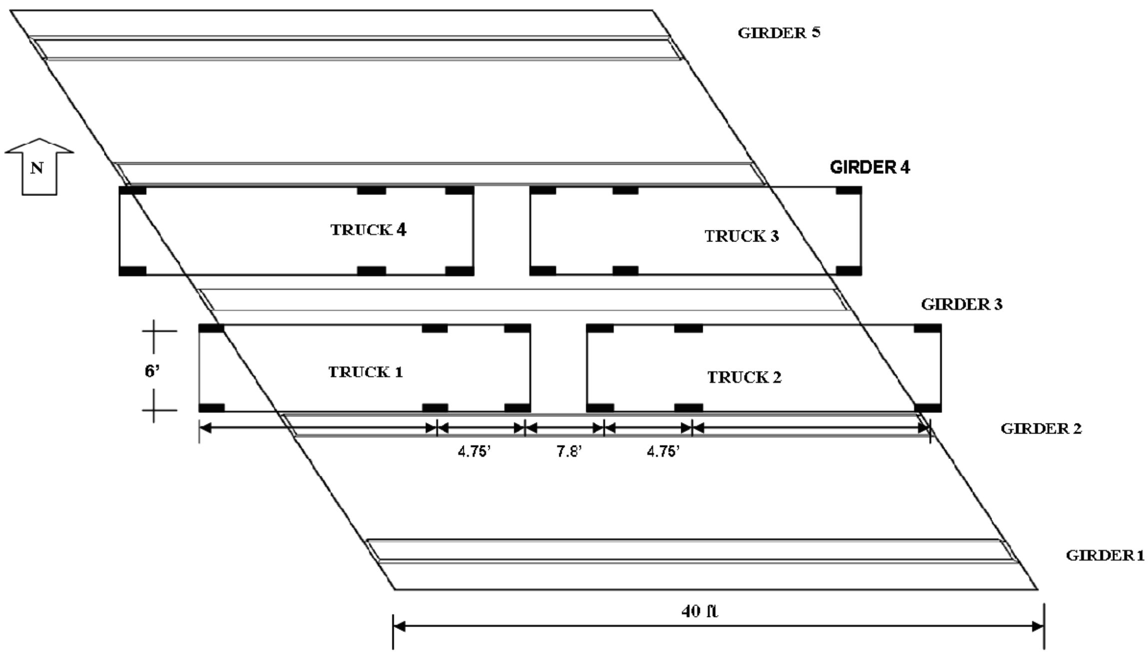

Figure 4.12 Locations of Test Vehicles on Test Span

Figure 4.13 2 Trucks (right), 4 Trucks (left)

4.5.2 Test Results and Comparison to FE Analysis

The cracks noted in Figure 4.6 showed no visible opening or extension during the course of the load tests of the structure. Following the load test, the recorded measurements were analyzed and were compared to those of the FE analysis. The readings were found to be in very good agreement with the FE analysis and are presented for the two and four truck load cases in Figure 4.13. This figure illustrates the distribution of load to the individual girders identified in Figure 4.9.

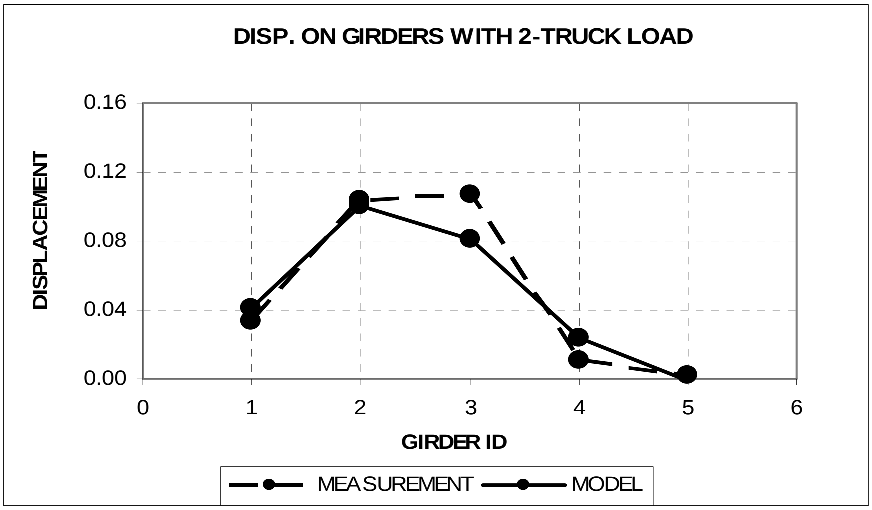

Figure 4.14 Girder Displacements Under 2 Truck Loading

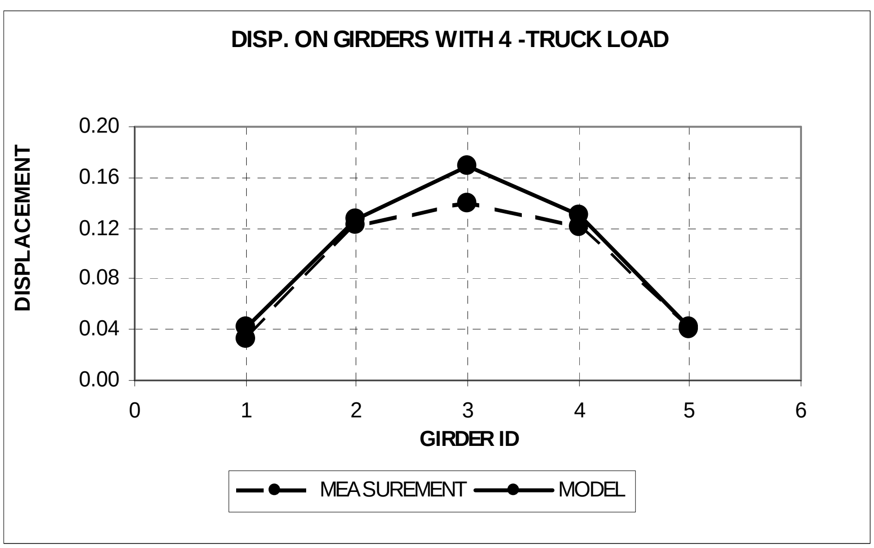

Figure 4.15 Girder Displacements Under 4 Truck Loading

Tables 4.4 and 4.5 present the information numerically from both the two-truck and four-truck load cases. The potentiometer readings for the gauges at girders 1, 4, and 5 are not reported due to their failure to report data. As with the previous bridge discussed in Chapter 3, the results in Figure 4.13 and Tables 4.4, and 4.5 show that a properly constructed FE analysis of this bridge captures the load-deflection relationship for the individual reinforced concrete girders as well as the load distributing pattern among the individual girders quite accurately.

4.5.3 Discussion and Conclusions

The calculated and measured deflections of girder 1 and girder 3 under the 4-truck load case differ by approximately 22% and 19% respectively, while the calculated and observed deflections of the other three girders are within 7% of each other. The differences between measured and FE analysis deflections for the 2-truck load case are slightly larger than that of the 4-truck load case. The differences for the 2-truck load case are as follows; 20% in girder 1, 2% in girder 2, 32% in girder 3, 54% in girder 4, and 400% in girder 5. The large percentage differences in girders four and five especially can be attributed to the magnitude of the deflections. During the 2-truck load case, these girders were very lightly loaded and the deflections were quite small, making their measurements approaches the level of sensitivity of the instrumentation. With the FE analysis in good agreement with the measured results, it is concluded that the FE model provides a reasonable representation of the bridge's overall performance.

Table 4.3 Test and FE Analysis Deflections – 2 Trucks

Potentiometers (PT) and Dial Gauges (DG)

	G1		G2		G3		G4		G5	
Test #	PT	DG	PT	DG	PT	DG	PT	DG	PT	DG
1	na	0.035	0.107	0.11	0.063	0.158	na	0.008	na	0.002
2	na	0.033	0.102	0.103	0.059	0.165	na	0.012	na	0.003
3	na	0.032	0.098	0.097	0.053	0.147	na	0.012	na	0.003
Measurement	**na**	**0.033**	**0.102**	**0.103**	**0.058**	**0.157**	**na**	**0.011**	**na**	**0.003**
Avg. Measured	**0.033**		**0.103**		**0.107**		**0.011**		**0.003**	
FEM	**0.041**		**0.101**		**0.081**		**0.024**		**-0.001**	

In terms of the overall performance of this bridge, the maximum measured deflection was 0.138 in (3.51 mm) under four trucks, each of which was approximately 1.85 times larger than the H-15 truck for which the bridge was designed. This maximum deflection is far less than the value 0.6 in (15 mm) that corresponds to L/800, the maximum deflection permitted by the AASHTO LRFD Bridge Design Specifications.

Table 4.4 Test and FE Analysis Deflections – 4 Trucks

Potentiometers (PT) and Dial Gauges (DG)

	G1		G2		G3		G4		G5	
Test #	PT	DG	PT	DG	PT	DG	PT	DG	PT	DG
1	na	0.029	0.129	0.123	0.155	0.118	na	0.118	na	0.038
2	na	0.031	0.130	0.117	0.157	0.118	na	0.118	na	0.040
3	na	0.036	0.127	0.123	0.157	0.122	na	0.122	na	0.042
4	na	0.032	0.124	0.125	0.152	0.124	na	0.124	na	0.042
Avg.	**na**	**0.032**	**0.127**	**0.122**	**0.155**	**0.121**	**na**	**0.121**	**na**	**0.041**
Avg. Measured	**0.032**		**0.125**		**0.138**		**0.121**		**0.041**	
FEM	**0.041**		**0.128**		**0.17**		**0.13**		**0.041**	

CHAPTER 5 FIELD TESTING OF A PRESTRESSED I-GIRDER BRIDGE

5.1 DESCRIPTION

5.1.1 Bridge Details

GDOT bridge ID # 223-0034-0 is located in Paulding County GA, and carries State Route 120 over Little Pumpkinvine Creek approximately 5 miles south of Dallas, GA. It was designed for HS-20 loading using the AASHTO 1989 specifications and was constructed in 1992. The main structural system consists of pre-stressed concrete I-girders arranged in four simply supported spans. Figure 5.1 shows an overview of the girder and bridge structure.

Figure 5.1 Bridge 223-0034-0 Overview

The bridge is 216 ft (65.8 m) long and is comprised of two 40-ft (12.2-m) Type II prestressed I-girder spans and two 68-ft (20.7-m) Type III prestressed I-girder spans (Figure 5.2). The bridge has a slight curve; however, the girders of the bridge are essentially perpendicular to the pier caps. Span 3 crosses over the creek, while the other spans all cross approximately 10 ft (3.05m) over the flood plane. The bridge has a deck width of 43¼ ft (13.2 m) and a roadway width of 40 ft (12.2 m). The 68-ft (20.7-m) spans are comprised of five type III I-girders that are composite with the 9⅛ in (232 mm) thick slab (Figure 5.3).

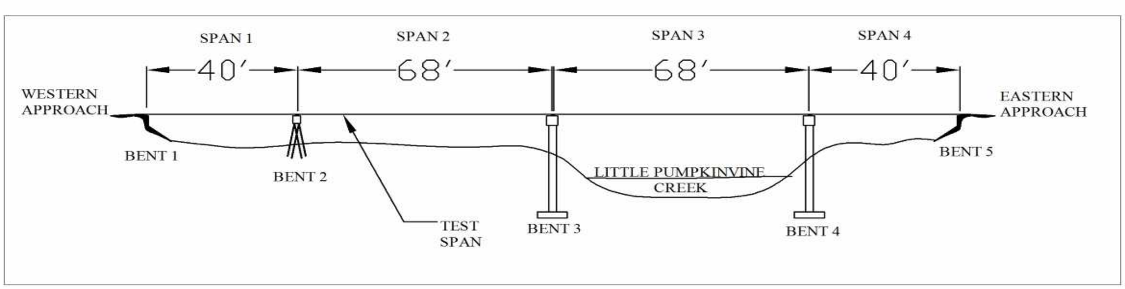

Figure 5.2 Bridge 223-0034-0 Elevation

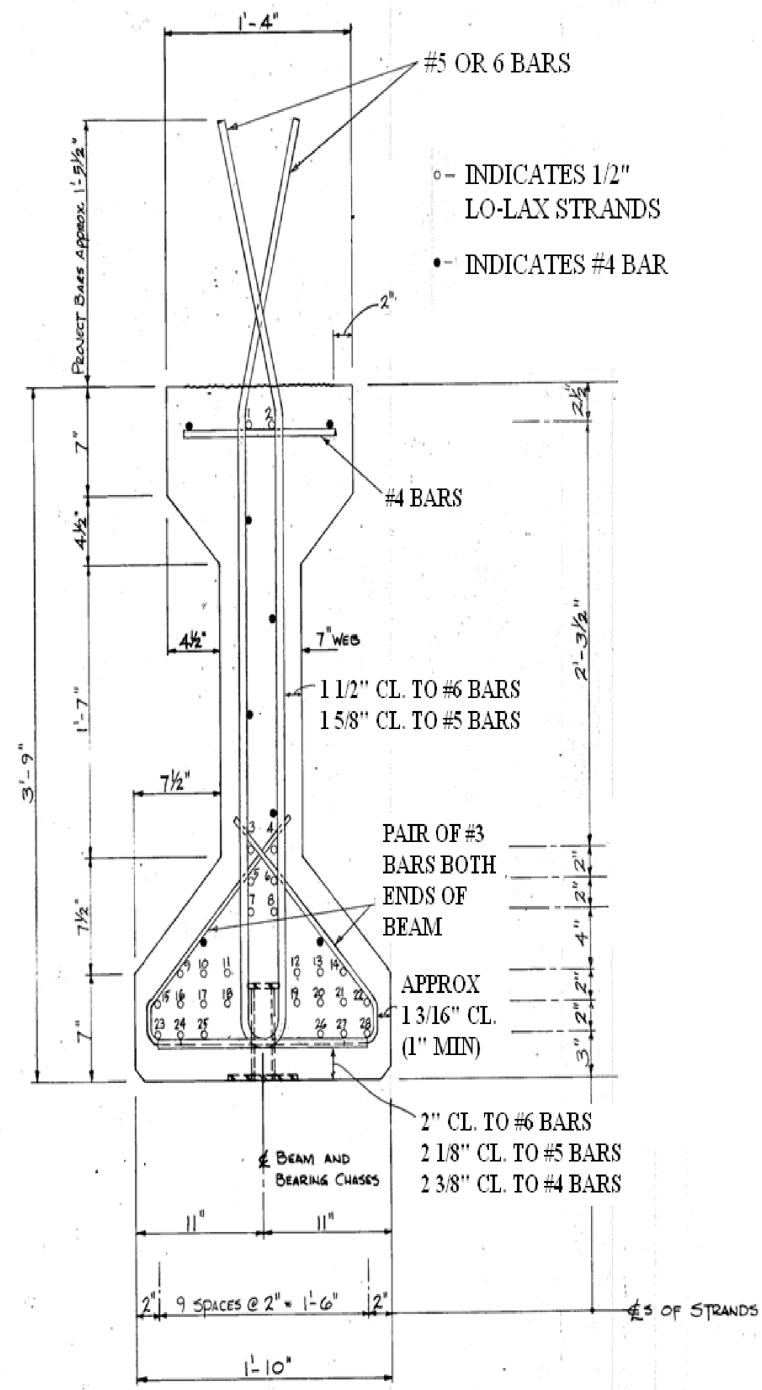

Figure 5.3 Type III Prestressed I-Girder Cross Section (Ref. GDOT Drawings)

The slab and pier caps are all constructed of 3,500 psi (24 MPa) concrete and reinforced with 60 ksi (414 MPa) steel reinforcing bars. The girders are constructed of 6,000 psi (41 MPa) concrete and a steel prestressing strand with an ultimate strength of 270 ksi (1,862 MPa). Figure 5.4 shows a cross section locating the reinforcement in the girders and slab.

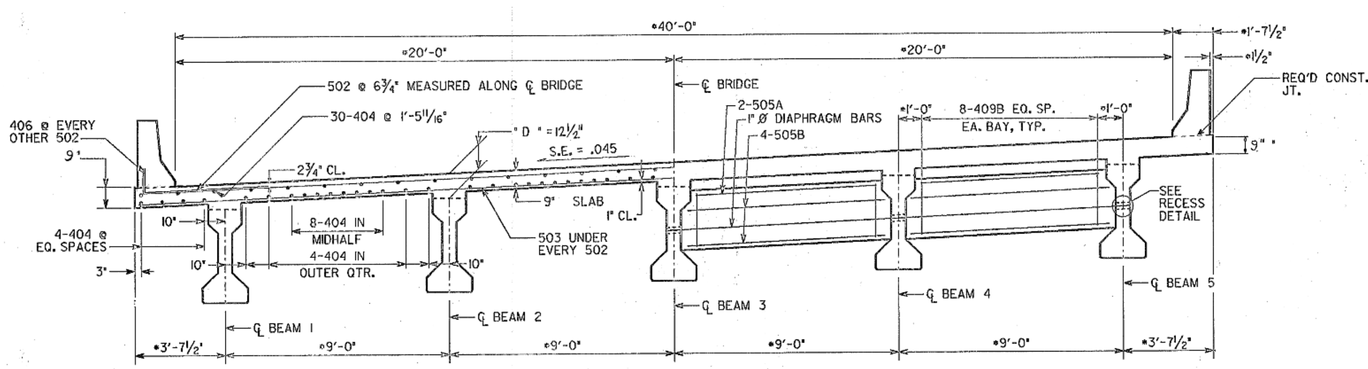

Figure 5.4 Superstructure Reinforcement Cross Section (Ref. GDOT Drawings)

Supporting the girders at bent 2 is a reinforced concrete pier cap measuring 52 in (1321 mm) wide by 27 in (686 mm) deep which, in turn, is supported by pairs of steel H12x53 piles located at each of the five girders. Bents 3 and 4 are comprised of a pier cap supported by two columns and a footing, all constructed of reinforced concrete. Finally, bents 1 and 5 are supported by a reinforced concrete an abutment founded on H-piles.

5.1.2 Bridge Condition

The bridge was last inspected on July 24, 2004. According to the GDOT inspector's report, the bridge deck has a condition assessment rating of 7. The GDOT inspector reported minor cracking in the deck over bents 2, 3, 4, and in spans 2 and 3 (see Figures 5.2 and 5.4). Minor spalling was reported in the supporting pre-stressed I-girders at bent 2, and the girders were assigned a condition rating of 7. The GDOT inspector judged the bents and abutments to be in good condition and gave then a condition rating of 7. The steel piles supporting bent 2 were in need of concrete encasement because they were in standing water during the inspectors' visit and exhibited minor corrosion.

5.2 RATING PROCEDURE

All condition ratings were determined at the time of the latest inspection. Based on this inspection, the GDOT engineer's calculations determined that the bridge did not required posting for either the legal loads or the HS-20 load.

Table 5.1 compares the GDOT engineer's rating results (using both the LFR and ASR rating methods, as is occasionally done for prestressed girder bridges) with the ratings determined independently by the research team in this study by the LFR and LRFR methods.[2] The results of the GDOT and independent ratings are very similar, but are not in total agreement. The GDOT rating uses different prestressed loss calculations than those detailed in the AASHTO Standard Specifications for Highway Bridges 2002.

[2] Details of the rating calculations are provided in Appendix E of the Report of Task 1.

The GDOT rating output file does not distinguish between the flexural strength check and allowable stress in serviceability check at the inventory rating level. The component rating factors for flexure and shear in the girders at Operating load levels show a rating reduction for LRFR of 33 and 38 percent respectively over that computed using LFR, as summarized in Table 5.1.

Table 5.1 Bridge Rating Results for HS-20 Vehicle (tons)

	Inventory			Operating	
Rating Method	Stress	Flexure	Shear	Flexure	Shear
LF (GDOT Calculations)	**41.5**		NA	NA	**85.9**
LF (Current Study)	**46.1**	55.4	51.5	92.5	**86.0**
	Design			Legal	
LRFR (Current Study)	NA	NA	NA	61.9	52.9

5.3 FINITE ELEMENT ANALYSIS

A finite element (FE) model of bridge 223-0034-0 was developed using ABAQUS, and was used to assist in the design of the load test and its instrumentation and obtain further insight into the performance of the bridge. The details of this analysis can be found in Appendix A.

5.4 INSTRUMENTATION PLAN

As with the bridges discussed in Chapter 3, and 4, the rating of this bridge was governed by the performance of the girders. Accordingly, the FE analysis was validated by measuring the deflections of each of the girders of span No. 2 (68 ft or 20.7 m) at the locations identified in Figures 5.5. The dial gauges and potentiometers was placed 3 ft (0.9 m) from mid-span. As with the previous bridges, the potentiometers were supported by a stable platform and a cable was hung from the bottom of the girders down to them (see Figure 4.11 – right photo). A dial gauge was also used at the center girder in order to check the potentiometer readings.

A new data acquisition system - the Modular Portable Data Logging and Alarming System (OMP-MODL) produced by Omega Engineering, Inc. - was used for this test. This system provided the capacity to tabulate and plot the instrument readings in real time with the aid of a laptop computer, and furnished 24 data channels as opposed to the 8 afforded by the DAC-Pro system used in the first two tests.[3] Since the OMP-MODL system had not been used in previous field tests conducted by personnel from the School of Civil and Environmental Engineering, it was decided to use the prestressed concrete bridge test as a trial run for the system. The trial run consisted of using the 5 potentiometers necessary to monitor displacements of the five prestressed I-girders and 19 DCTH Series DC to DC Linear Variable Differential Transformer (LVDT) Displacement Transducers produced by RDP Electronics Ltd that had been installed to

[3] This new system was acquired because it was anticipated that the additional channels would be required for the test of the steel girder bridge (described in Chapter 6), which involved monitoring shear strains in the concrete pier cap as well as the deflections and flexural strains in the steel girders.

monitor strain in the girders as well as shear at one location in the pier cap. Prior to field testing all deflection measuring devices were calibrated in the Structural Engineering and Materials Laboratory by the method presented in Section 3.4.2. During the test, however, it was observed that the signal-to-noise ratio from the LVDTs was too large to produce meaningful results. The calibration had not identified up the noise problem because the increments of displacement used during calibration were 0.1 in (2.54 mm), whereas during testing the gauges only experienced 0.0001 in (0.000254 mm) of displacement. Following the load test of this bridge, further calibration was performed and the noise level of the LVDTs was reduced from 1000με (microstrain) to +/- 10με by reducing the input voltage range of the OMP-MODL data acquisition system from +/- 5 Volts to +/- 2 Volts. This reduced range meant the OMP-MODL system would only pick up the displacement of the LVDTs in the center 40% of their full range; however, the test of the steel bridge in Chapter 6 was not expected to produce strains outside that range.

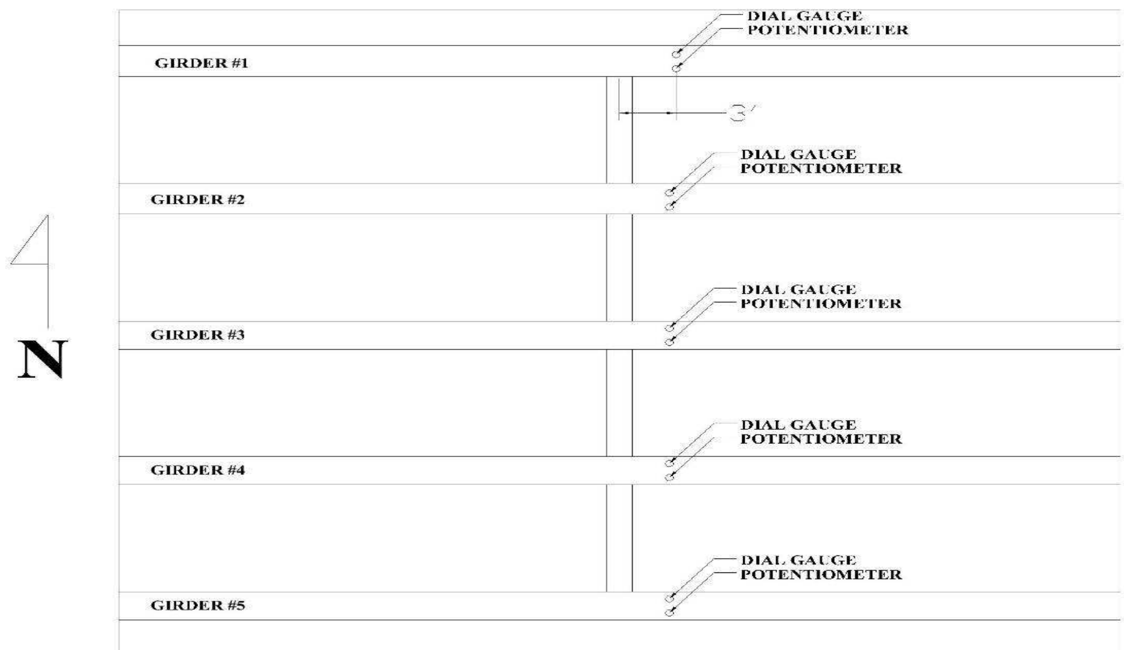

Figure 5.5 Instrumentation Location

5.5 DIAGNOSTIC LOAD TEST RESULTS

5.5.1 Testing

The load test was conducted on May 8, 2007 on span 2 (Figure 5.2). The loading on the bridge consisted of four GDOT tandem axle dump trucks (Figure 1.2 Type 3 truck) that were weighed individually before arriving at the bridge site. The individual truck and axle weights are summarized in Table 5.2.

Each test repetition involved two load cases. In the first, trucks 1 and 2 were placed on the south lane of the span in the locations noted in Figure 5.6; in the second all four trucks were placed on the span. Three repetitions of each load test were performed. The bridge was closed during each of the loading repetitions and reopened to traffic between repetitions to prevent traffic congestion. Each repetition began with a zero load reading; subsequently, the four trucks were backed onto the middle of the instrumented span, one truck at a time, starting with truck #1 and ending with #4 (Figure 5.6). Readings were continuously recorded using the OMP-MODL instrumentation system and manually recorded for the mechanical dial gauge. Finally, the trucks were moved off the test span and another zero reading was recorded before traffic was permitted to resume.

5.5.2 Test Results and Comparison to FE Analysis

The results of the two-truck tests using trucks 1 and 2 are shown in Table 5.3 and Figure 5.7. Overall, these results are in good agreement with those of the FE analysis and clearly show the concentration of loads on one side of the bridge. The readings from the potentiometer and dial gauge located at girder number 3 are in very good agreement, with a difference of only 5%. The girder 1 potentiometer had the largest discrepancy from the FE analysis, measuring 0.011 in (0.31 mm) vs. the FE analysis result of 0.25 in (6 mm). It is probable that the potentiometer on girder 1 was not operating properly during the third repetition, as the measured deflection is an order of magnitude less than in the two previous trials. If this third measurement is discarded, the measured deflection of girder 1 (average of repetitions 1 and 2) differs by 47% from the FE analysis. As a result the measurements from girder 1 during the third loading and girder 2 during the first loading have been considered as outlier and not included in the measured average. Unfortunately, the potentiometer on girder 5 failed during testing and provided no data.

The second test load configuration involved all four trucks. The results from this four-truck load configuration are presented in Table 5.4 and Figure 5.8. Again, the largest difference between the FE analysis and measured results (average of three measurements) is in the edge girder 1 at 29% while girder 2 is below 10% difference, and 3 and 4 are under 5% different from the FE analysis displacements. As with the two-truck load case, the deflections measured by potentiometer and dial gauge at girder 3 are in excellent agreement, differing by less than 1%.

Table 5.2 Truck Weight (lb) Details for Paulding County Bridge Test

	Load on Axel 1	**Load on Axel 2**	**Load on Axel 3**	**Overall Truck Weight**
TRUCK 1	18,600	17,000	16,400	52,000
TRUCK 2	18,500	17,000	16,700	52,200
TRUCK 3	18,900	16,900	16,200	52,000
TRUCK 4	18,300	16,900	16,500	51,700

5.5.3 Discussion and Conclusions

The difference between calculated and observed deflections for the four-truck load case was very small. As with the previous two bridges tested, the larger percentage difference in the two-truck load case can be attributed to the decreased magnitude of the deflections, while the accuracy of the sensors remained constant. With the FE analysis results in good agreement with the measured results, it may be concluded that the FE analysis practice employed to model the performance of this bridge analytically is sufficiently accurate to be employed with confidence to assess other pre-stressed bridges in the Georgia bridge inventory.

Unlike the other three bridges analyzed and tested in this project, this prestressed girder bridge was designed for HS-20 loading. The HS-20 vehicle weighs a total of 72,000 lbs (320 kN). The test vehicles, with a combined total weight of 207,900 lbs (924 kN), exceeded the design vehicle weight, yet only resulted in a maximum bridge deflection of 0.206 in (5.2 mm). This deflection is only 20% of the deflection limit of L/(800) limit [1.02 in or 5.2 mm] stipulated by AASHTO's LRFD specification. In terms of its overall performance, the bridge remained well within its elastic limit during the test despite being loaded well beyond the design level.

Table 5.3 Test and FE Analysis Deflections – 2 Trucks

2 Trucks	Potentiometers					Dial Gauge
Test #	G1	G2	G3	G4	G5	G3
1	0.0071	0.0049*	0.1026	0.1185	-	0.107
2	0.0141	0.0318	0.1258	0.1262	-	0.114
3	-0.0006*	0.0425	0.1159	0.1044	-	0.106
Measurement	**0.011**	**0.037**	**0.115**	**0.116**		**0.109**
FE Analysis	**0.020**	**0.037**	**0.120**	**0.120**	**0.074**	**0.120**

*Outlier – Omitted from average

Table 5.4 Test and FE Analysis Deflections – 4 Trucks

4 Trucks	Potentiometers					Dial Gauge
Test #	G1	G2	G3	G4	G5	G3
1	0.0773	0.1535	0.2205	0.1725	-	0.218
2	0.0622	0.1549	0.2195	0.1650	-	0.204
3	0.0550	0.1699	0.1845	0.1622	-	0.200
4	0.0659	0.1491	0.2005	0.1944	-	0.198
Measurement	**0.065**	**0.157**	**0.206**	**0.174**		**0.205**
FE Analysis	**0.091**	**0.171**	**0.215**	**0.181**	**0.1086**	**0.215**

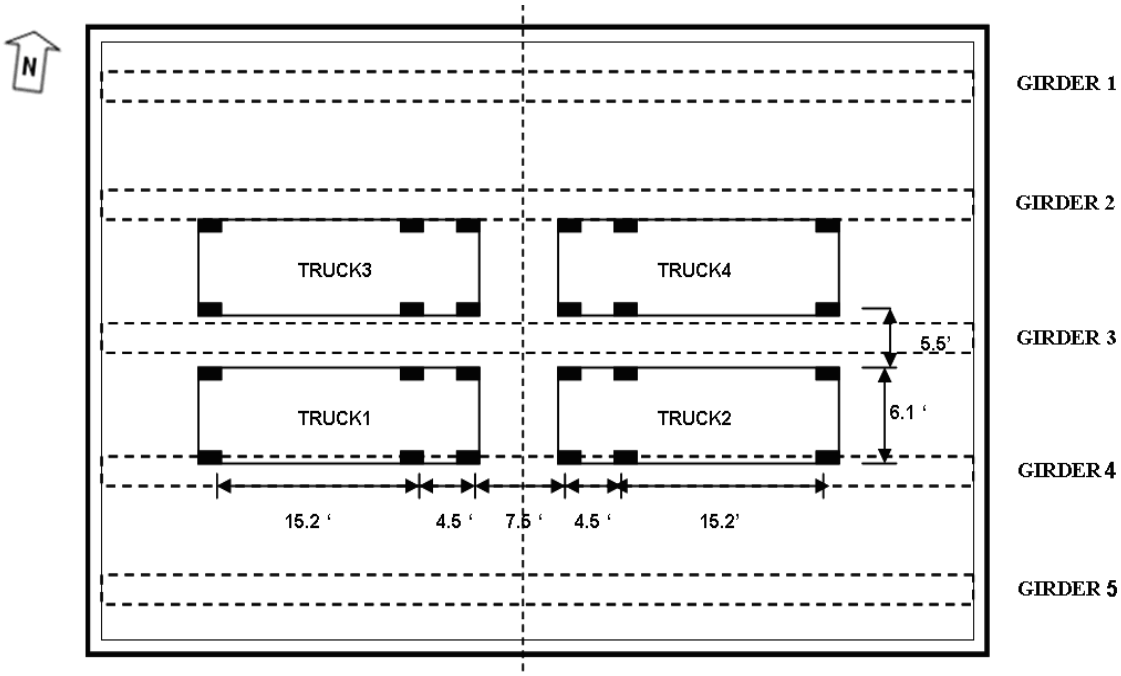

Figure 5.6 Locations of Test Vehicles on Test Span

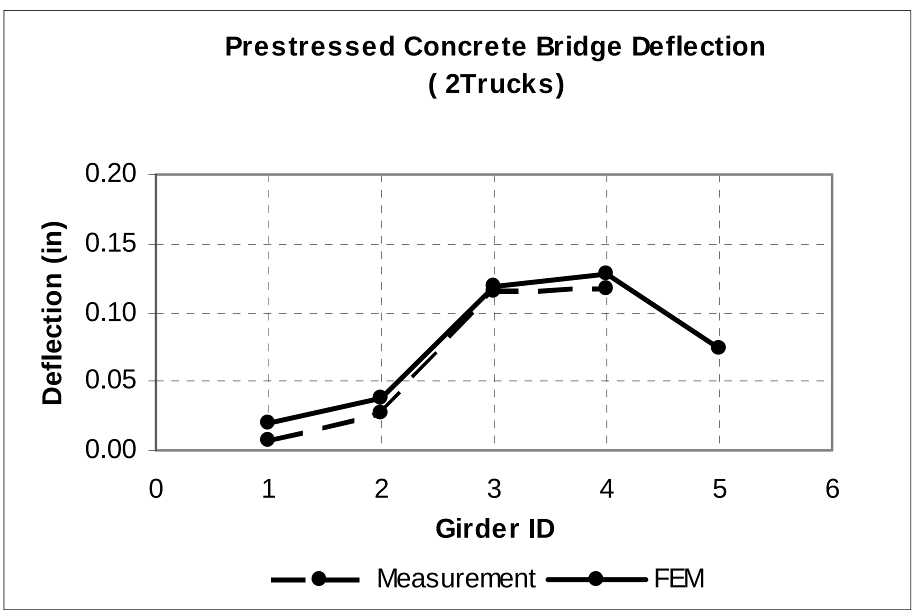

Figure 5.7 Girder Displacements Under 2 Truck Loading

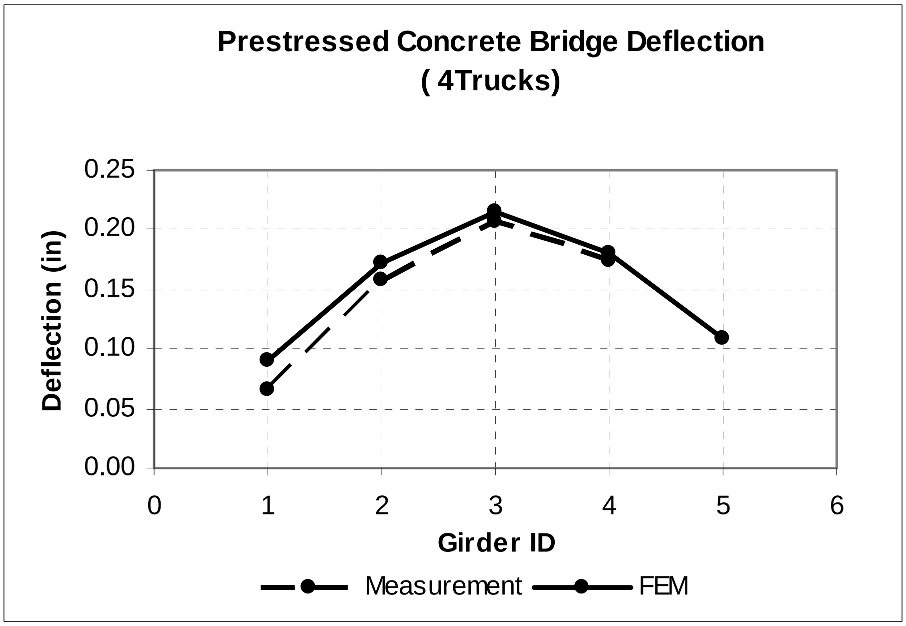

Figure 5.8 Girder Displacements Under 4 Truck Loading

CHAPTER 6 FIELD TESTING OF A STEEL GIRDER BRIDGE

6.1 DESCRIPTION

6.1.1 Bridge Details

This bridge is located in Dawson County GA (GDOT ID# 085-0018-0), and carries State Route 136 over the Etowah River approximately 5.7 miles east of Dawsonville. The bridge has four simply supported spans. Figure 6.1 shows an overview of the bridge, girders and supporting substructure. An elevation is shown in Figure 6.2. The bridge is posted, as described subsequently.

Figure 6.1 Bridge 085-0018-0 Overview

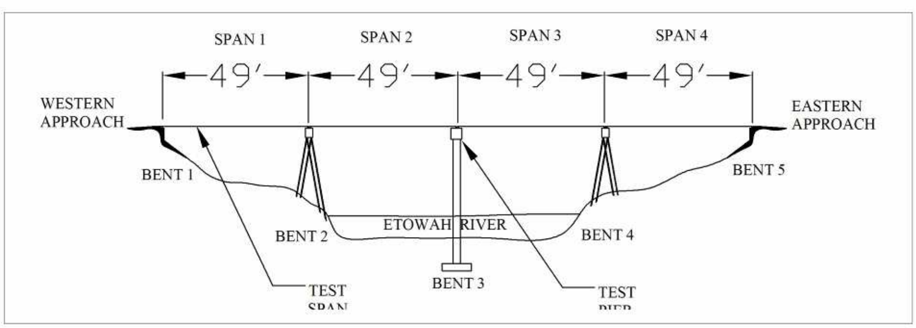

Figure 6.2 Bridge 085-0018-0 Elevation

This bridge was designed using the AASHTO 1961 specification, with interim revisions through 1963 for H-15 loading, and was constructed in 1965. The bridge is 196 ft (59.7 m) long and its four 49 ft (12.2 m) spans are supported by four steel W-shape girders spaced 8 ft (2.44 m) on center, with a full-depth diaphragm located at mid-span (Figure 6.3). The two outer girders are W33x118 sections while the two interior girders are W33 x 130 sections. The girders are perpendicular to the pier caps. The two end spans are 15 to 25 ft (4.57 to 7.62 m) above a sloping flood plain, and the center two spans cross approximately 35 ft (10.67 m) over the river. The bridge has a deck width of 32 ft (9.75 m) and a roadway width of 26 ft (7.92 m). The drawings show the steel girders to be non-composite with the 6.5 in (165 mm) thick slab (Figure 6.3).

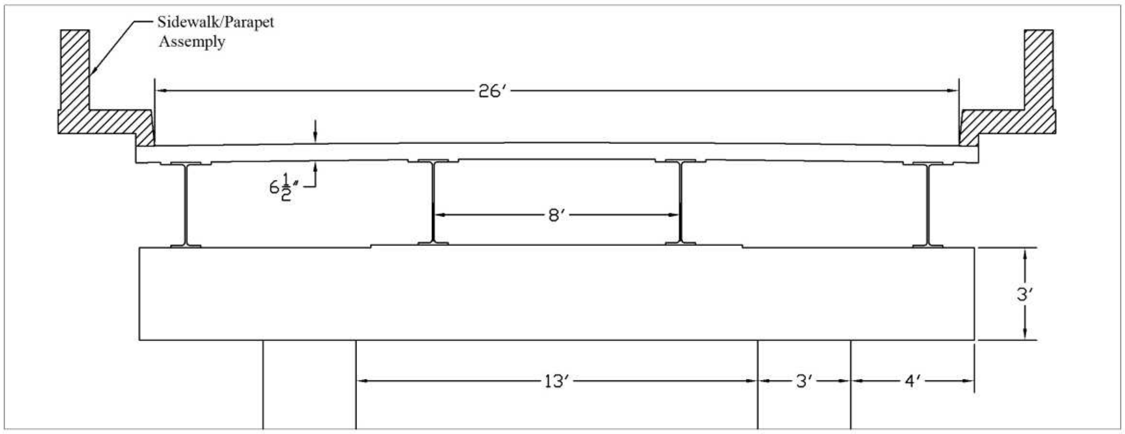

Figure 6.3 Girder and Slab Cross Section (Ref. GDOT Drawings)

The reinforced concrete slab and the pier caps are all constructed of 3,000 psi (20.7 MPa) concrete and reinforced with 40 ksi (276 MPa) steel bars. Supporting the girders at bents 2 and 4 identified in Figure 6.2 and shown in Figure 6.4 are reinforced concrete pile caps 36 in (914 mm) wide by 24 in (609 mm) deep, supported by two battered steel H 12x53 piles at the location of each of the four girders (Figure 6.1).

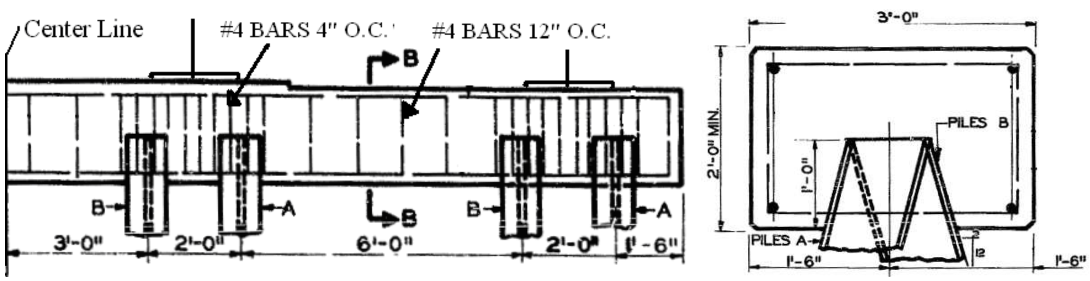

Figure 6.4 Pier Cap Elevation (left), Cross Section B-B(right) (Ref. GDOT Drawings)

Bent 3, shown in Figure 6.2, is comprised of a pier cap supported by two columns and footings, all constructed of reinforced concrete (Figure 6.5). The pier cap is 36 in (914 mm) square with top and bottom steel reinforcement and stirrups 4 in (102 mm) on center under each girder, labeled G1 through G4 in Figure 6.5, and 12 in (305 mm) on center elsewhere. The load on the pier cap comes from the four girders labeled G1 to G4, each being centered 2 ft 6 in (762 mm) from the face of the nearest column, and from the self-weight of the pier cap. The columns are 36 in (914 mm) square; each column is supported by a 6 ft (1.83 m) square footing detailed in Figure 6.5.

Finally, bents numbered 1 and 5 are supported by a reinforced concrete abutment founded on H-piles.

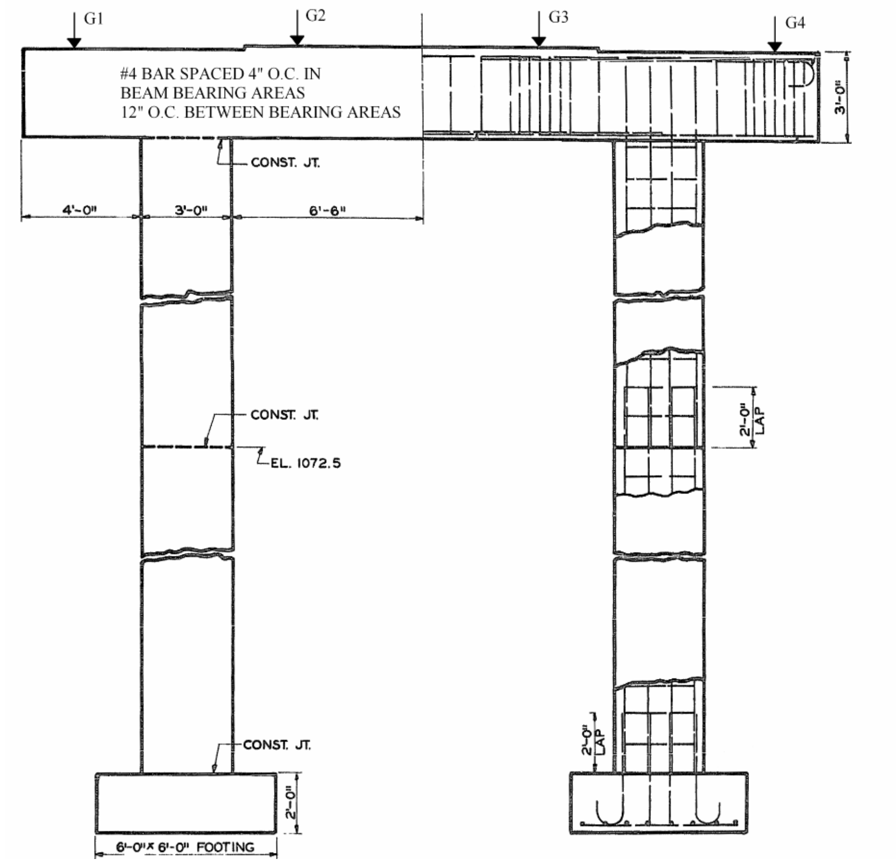

Figure 6.5 Reinforced Concrete Pier (Ref. GDOT Drawings)

6.1.2 Bridge Condition

The bridge was last inspected on June 30, 2005, and according to the GDOT inspector's report, the deck and substructure were assigned a condition assessment rating of 6. The inspection report indicates that there is spalling, aggregate exposure, and transverse cracking in the deck in all spans, and recommends action to seal transverse cracks in two of the spans and deck spalls where rebars are exposed. The inspector found the steel girders to be in good condition, with the exception of minor deflections and a fractured anchor bolt at bent 2. During instrumentation prior to load testing, it was observed that the girders had a good coating of protective paint with only minor corrosion and rust spots near their supports. During grinding prior to installing instrumentation, no sign of rust or corrosion under the protective paint was observed on the girder webs and flanges. On the other hand, it was noted that several anchor bolts were severely corroded, including the bolt that was found to be fractured by the GDOT inspector (Figure 6.6). The GDOT inspector also gave the substructure an assessment rating of 6.

Figure 6.6 Corrosion and Fracture of Anchor Bolt

6.2 RATING PROCEDURE

All condition assessment ratings were determined at the time of the latest inspection on June 30, 2005. Based on that inspection, the GDOT engineer determined that the bridge required posting for 5 of the 6 legal loads. The resulting postings restrict H type trucks to a maximum of 21 tons, Tandem trucks to 23 tons, Logging trucks to 27 tons, 3-S-2 type trucks to 32 tons, and HS type trucks to 25 tons (Figure 6.7).

Figure 6.7 Posting Sign for Bridge 085-0018-0

LFR AS and LRFR ratings of this bridge were performed in this study for the superstructure and substructure[4] using the HS-20 loading case only; that rating was found to be in good agreement with the GDOT rating results (see Tables 6.1 and 6.2). The portion of the substructure evaluated was the reinforced concrete pier cap between spans 2 and 3; that substructure was found to govern the bridge's rating. Table 6.1 compares the GDOT superstructure ratings with the independent LFR, ASR, and LRFR-based results obtained in this study, while Table 6.2 compares the same ratings performed on the substructure. The ratings of both the governing limit state (shear in the pier cap) and the capacity of the girders using LRFR and LFR differ by 8% in the superstructure rating and a 4% difference in the substructure rating. Note that the HL-93 load is used with the Design ratings in LRFR instead of the HS-20 load so is omitted from the tables. The Legal loads (Figure 1.1) are used for both the LRFR Legal rating category and the ASR and LFR Operating rating category, and determine the posting values for the bridge.

Table 6.1 Superstructure Rating in Tons for HS-20 vehicle

	Inventory	Operating
Failure Mode	Flexure	Flexure
AS (Current Study)	29.5	47.9
LF (GDOT Calculations)	**24.5**	**40.9**
LF (Current Study)	**25.6**	**42.5**
	Design	Legal
LRFR (Current Study)	Not Comparable	38.9

Table 6.2 Substructure Rating in Tons for HS-20 vehicle

	Inventory	Operating
Failure Mode	Shear	Shear
AS (Current Study)	9.36	27.0
LF (GDOT Calculations)	**14.8**	**24.7**
LF (Current Study)	**15.5**	**25.9**
	Design	Legal
LRFR (Current Study)	Not Comparable	27.0

A closer examination of the substructure reveals that the shear span-to-depth ratio of the loads and reactions in the pier cap is less than one. The distance from the center of the girder that transfers load to the pier cap and the face of the column that supports the pier cap is only 2.5 ft (762 mm) while the depth of the pier cap is 3 ft (914 mm). As a result, the pier cap behaves structurally as a deep beam and as such Section 5.8.3.4.1 Simplified Procedure for Nonprestressed Sections of AASHTO's *Manual for Condition*

[4] Details of the rating calculations are provided in Appendix F of the Report of Task 1.

Assessment and Load and Resistance Factor Rating of Highway Bridges (2005) should not be used. Section 5.6.3 (page 5-25) of the same manual makes provisions for the analysis of such deep beams using the strut and tie method, but the older LF rating method uses the traditional ACI 318 approach to calculating shear capacity. Using a strut and tie analysis results in a more realistic assessment of pier cap capacity; of particular significance, the estimated capacity is considerably higher than what would be calculated by Section 5.8.3.4.1 as shown in Table 6.3.

Table 6.3 Estimated Pier Capacity (Tons) Using AASHTO's LRFD 2005 Manual

	Legal
LRFD Manual Equation C5.8.3.3-1 Section 5.8.3.4.1	27.0
Strut and Tie Analysis Section 5.6.3	34.0

Incorporating this more appropriate capacity computation, the revised ratings for shear in the pier cap in Table 6.4 are compared to the current state rating and to the rating computed using the FE analysis (described subsequently). These revised ratings result in a 26% increase in the posted load for the HS-20 Truck using the LRFR provision for strut and tie models. With the 26% increase in the rating capacity of the pier cap, shear in the pier cap no longer governs the rating of this particular bridge; however, this may not be the case for all Georgia bridge structures in which pier cap shear capacity governs the bridge rating. Since the strut and tie analysis increased the computed capacity of the pier cap, the posting load for each of the legal trucks will be increased.

Table 6.4 Substructure Rating in Tons for HS-20 vehicle

	Inventory	Operating
Failure Mode	Shear	Shear
LF (GDOT Calculations)	14.8	24.7
	Design	Legal
LRFR (Current Study) (Strut-and-Tie Analysis)	N.A.	34.0
FE Analysis (Current Study)	N.A.	44.6

6.3 FINITE ELEMENT ANALYSIS

Finite element analyses were undertaken to obtain further insight into the performance of the bridge and to further examine the strut and tie behavior of the concrete pier cap at bent 2. The finite element analyses were performed by means of the commercially available ABAQUS software; details are presented in Appendix A. Preliminary FE analyses using anticipated test vehicle loads were performed prior to testing. The results of these analyses were then used as a basis for designing instrumentation and monitoring the bridge response during the load test.

6.4 INSTRUMENTATION PLAN

With the pier cap shear capacity governing the bridge's rating and questions as to which AASHTO equations to use in its analysis, the pier cap instrumentation was viewed as critical in this test. Strain in the pier cap was measured using a set of three Linear Variable Differential Transformers (LVDTs) arranged in a rosette (Figure 6.8) at each location where strain was to be measured. The pier cap was instrumented with five of these LVDT rosettes and 2 LVDTs mounted horizontally on the pier cap over the supporting columns. Each LVDT had a gauge length of 10 in, measured as the distance between the centers of the mounts at each end of the LVDT (Figure 6.9). The expected flow of forces according to strut-and-tie and the FE analysis is shown in Figure 6.10, and was used to design the placement of LVDTs in the pier cap. Since the magnitude of the strain was expected to be very small, the orthogonal LVDTs in each rosette were placed to be coincident with the directions of principal tension and principal compression; the third is at a 45 degree angle with respect to the other two. Each of the five rosettes was placed, as shown in Figures 6.11 and 6.12, in a compression strut between the girder (load) and column (support).

Due to the height of the pier cap, the instrumentation was performed using a bucket truck provided by the GDOT. As a result, a GDOT bridge inspector was also present during the instrumentation to operate the bucket truck; he identified several areas of delamination (Figure 6.13) and general deterioration of the concrete in the pier cap, but found it to be structurally sound with no apparent cracking. His assessment of the bridge was in good agreement with the latest inspection on record with the GDOT. He identified the fractured anchor bolts as in need of replacement, and found the girders in the instrumented spans to be in good condition.

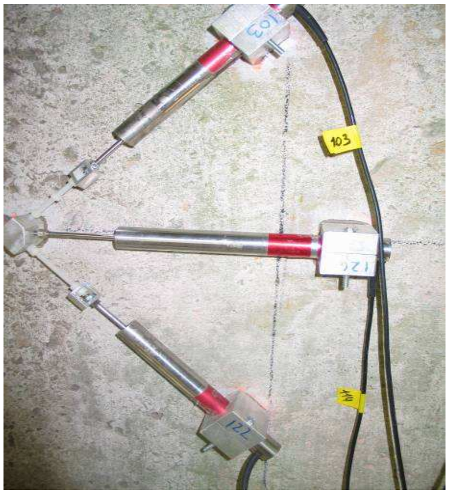

Figure 6.8 LVDT Rosette

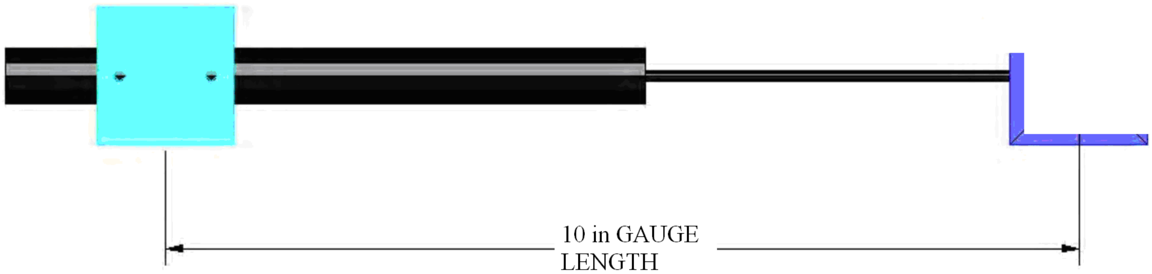

Figure 6.9 LVDT Mounts

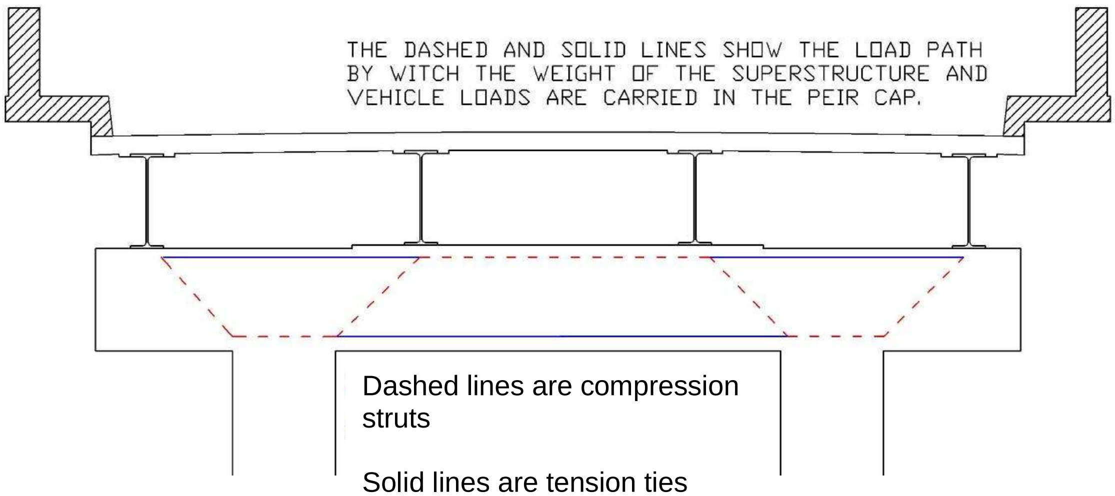

Figure 6.10 Load Paths within Pier Cap

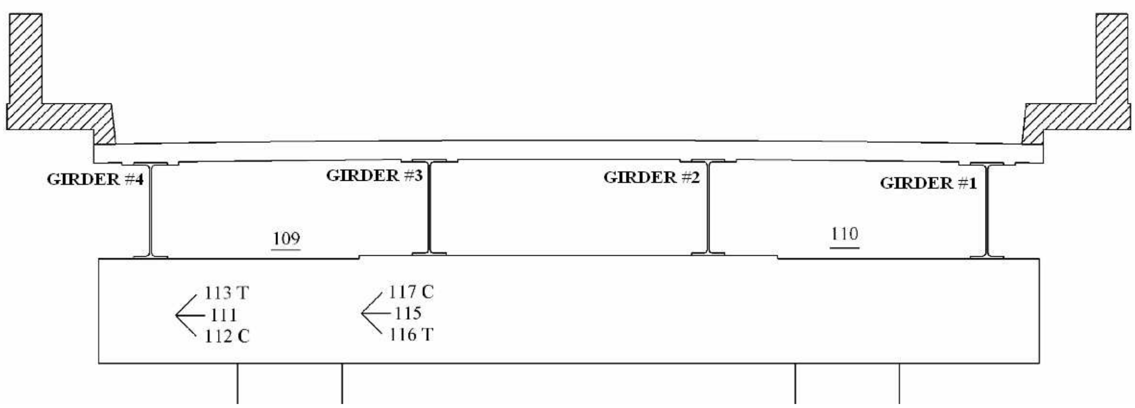

Figure 6.11 Pier Cap Instrumentation on West Face

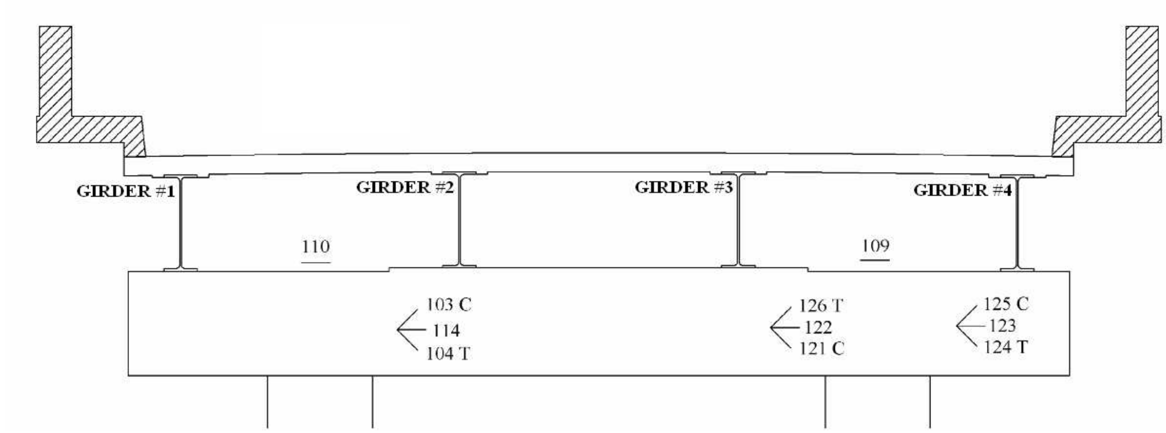

Figure 6.12 Pier Cap Instrumentation on East Face

Figure 6.13 Delamination of Concrete Pier Cap

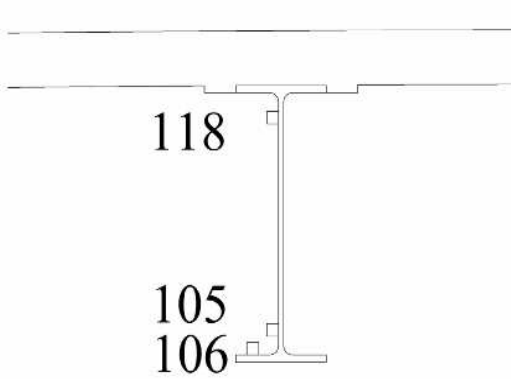

Figure 6.14 LVDT Location on Girder #2

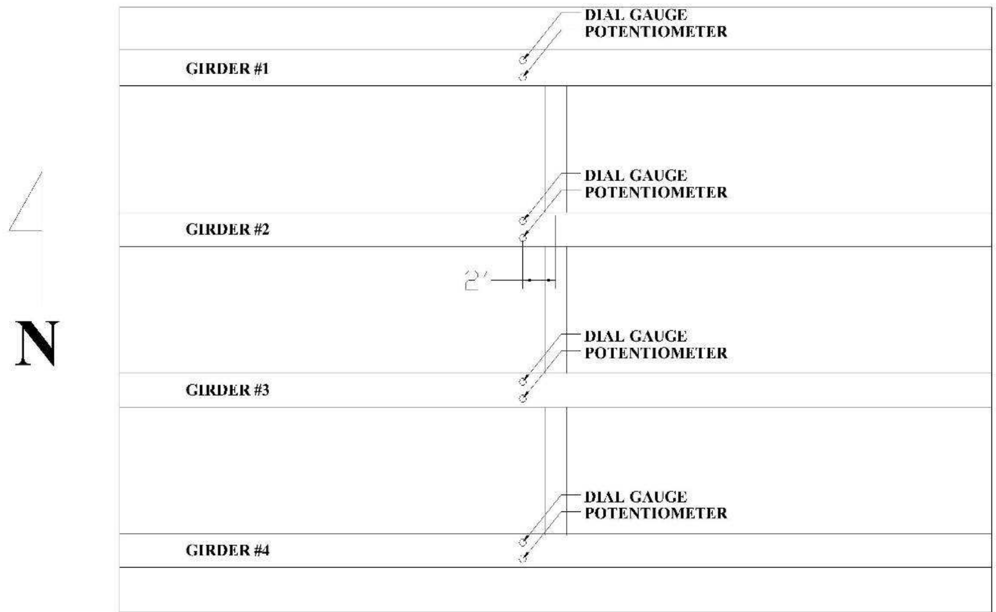

Figure 6.15 Instrumentation Location

In addition to the LVDTs installed on the pier cap, LVDTs were also installed on one of the bridge's steel girders to measure strain through the depth of the girder (see Figure 6.14).

Potentiometers were placed under each of the steel girders to measure their deflections, similar to the procedure described in the preceding chapters. The instrumentation was located 2 ft (0.61 m) from the concrete diaphragm at midspan, as shown in Figure 6.15. Each potentiometer and LVDT used in the test was calibrated in the Structures Laboratory of the School of Civil and Environmental Engineering, Georgia Institute of Technology, as described in Chapter 3.

The Modular Portable Data Logging and Alarming System (OMP-MODL) system described in Section 5.4 was used for recording the deflection and strain measurements during the load test of this bridge. This system had been purchased specifically for the test of the steel girder bridge in order to have a sufficient number of data channels to monitor the strains in the pier cap and girder deflections. The MODL system, with its 24 data channels instead of the 8 afforded by the DAC-Pro system used in the earlier tests of the reinforced concrete bridges described in Chapters 3 and 4, provided the capacity to tabulate and plot the instrument reading in real time with the aid of a laptop computer. During calibration the noise level of the LVDTs was reduced from 1000 micro strain to +/- 10 micro strain through reducing the input voltage range of the OMP-MODL data acquisition system from +/- 5 Volts to +/- 2 Volts. The reduced input range meant the OMP-MODL system would only pick up the displacement of the LVDTs in the center 40% of their full range; however, none of the measurements during the tests of the steel bridge exceeded this range.

6.5 DIAGNOSTIC LOAD TEST RESULTS

6.5.1 Testing

The load test was conducted on May 10, 2007. The test was performed in two stages; the first stage was intended to measure the deflection of span 1 (Figure 6.2) and the second stage was intended to maximize the shear in the pier cap of bent 3 (Figure 6.2). The test loading was applied using 4 GDOT tandem axle dump trucks that had been weighed individually before arriving at the bridge site; the axle and total weights for these test vehicles are presented in Table 6.5. The first-stage test of span 1 was performed with the trucks positioned as shown in Figure 6.16. After this stage was completed, the second-stage test of the pier cap was performed using the vehicle configuration shown in Figure 6.17. Four repetitions of the span loading and two repetitions of the pier cap loading were performed with the bridge closed during each loading repetitions and reopened to traffic between repetitions to reduce traffic congestion. Each repetition began with a zero load reading; subsequently, all four trucks were backed onto the middle of the instrumented span or over the pier cap at once. Readings were then recorded from the OMEGA instrumentation system, the trucks were moved off the bridge, and traffic was permitted to resume.

Table 6.5 Truck Weights (lb) for Etowah River Bridge Test

	Load on Axle 1	Load on Axle 2	Load on Axle 3	Overall Truck Weight
TRUCK 1	20,200	22,300	20,900	63,400
TRUCK 2	17,300	22,700	22,300	62,300
TRUCK 3	20,500	23,000	21,500	65,000
TRUCK 4	21,600	23,700	23,400	68,700

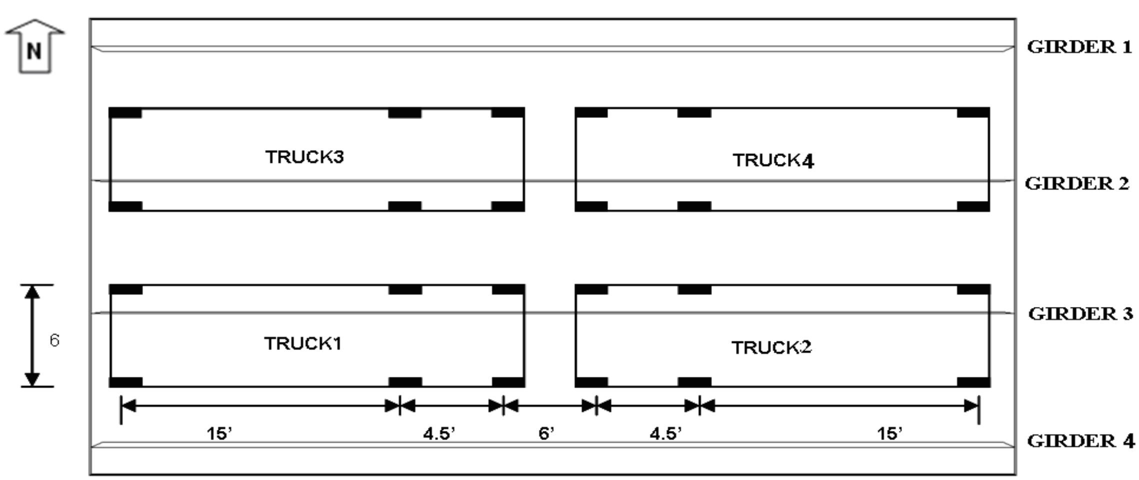

Figure 6.16 Locations of Test Vehicles During Testing of the Span #1

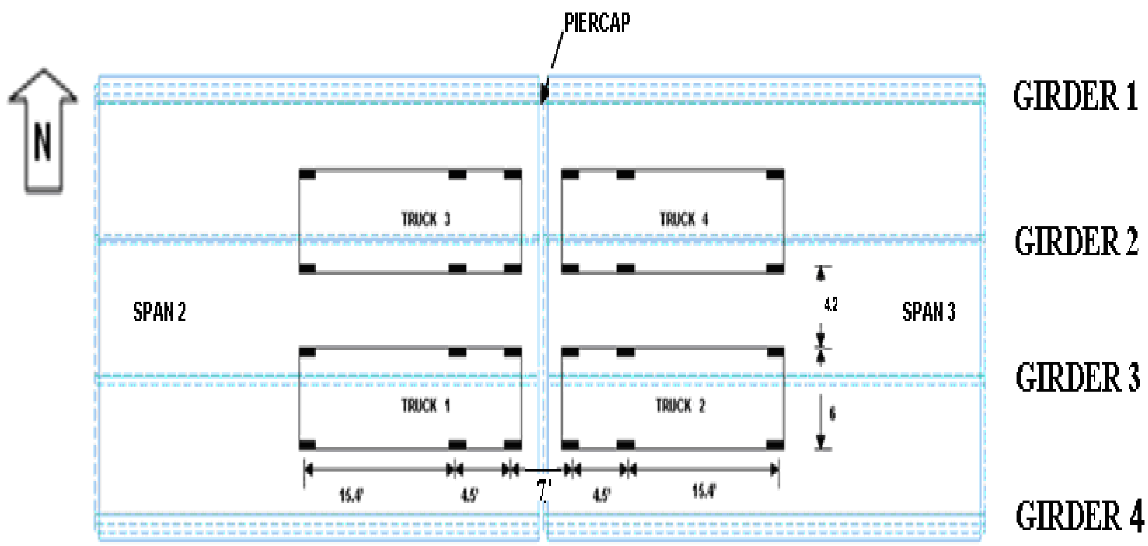

Figure 6.17 Locations of Test Vehicles During Testing of the Pier Cap

6.5.2 Test Results and Comparison to FE Analysis

6.5.2.1 Girder Strains

The strain results from the test of span 1 are presented in Figure 6.18 and Table 6.6. Gauge # 118 did not function properly and the test cannot be used to confirm whether any composite action exists between the girders and slab. The strains measured from the other two LVDTs are quite close for all three repetitions, indicating that the test procedure leads to reproducible results.

6.5.2.2 Girder Deflections

Table 6.7 presents a comparison of the span 1 girder deflections measured during four repetitions of the load test to the results of the FE analysis of the bridge. The deflections are reproducible from test to test; those deflections measured by the potentiometers during the testing of span 1 are all within 15 to 25% of the FE analysis deflections. The comparison of these results can be visualized in Figure 6.17, illustrating the distribution of forces to the individual girders. While the differences between the analytical results and *in situ* measurements are well within the range of what would be expected in such a comparison, the agreement is not as good as that obtained for the two reinforced concrete bridges tested earlier (summarized in Chapters 3 and 4). The difference in the values is believed to stem from the presence of a degree of composite action between the girders and the slab in the elastic range of bridge response. Inspection of the bearings over the pier caps yielded some signs of corrosion that may have had some small contribution to locking of those joints (Figure 6.6), but such an inspection of the abutment support was not performed because the grade of the slope leading up to it made it inaccessible.

Table 6.6 LVDT Strain Test Results (+/-10 Micro Strain)
(Span #1 – Girder #3)

Gauge #		118	105	106
4 Trucks	Trial 1	na	-178.4	-194.0
	Trial 2	na	-193.6	-194.6
	Trial 3	na	-210.5	-210.9
	Trial 4	na	-190.9	-185.9
	Average	**na**	**-194.2**	**199.8**
	FE Analysis			**287**

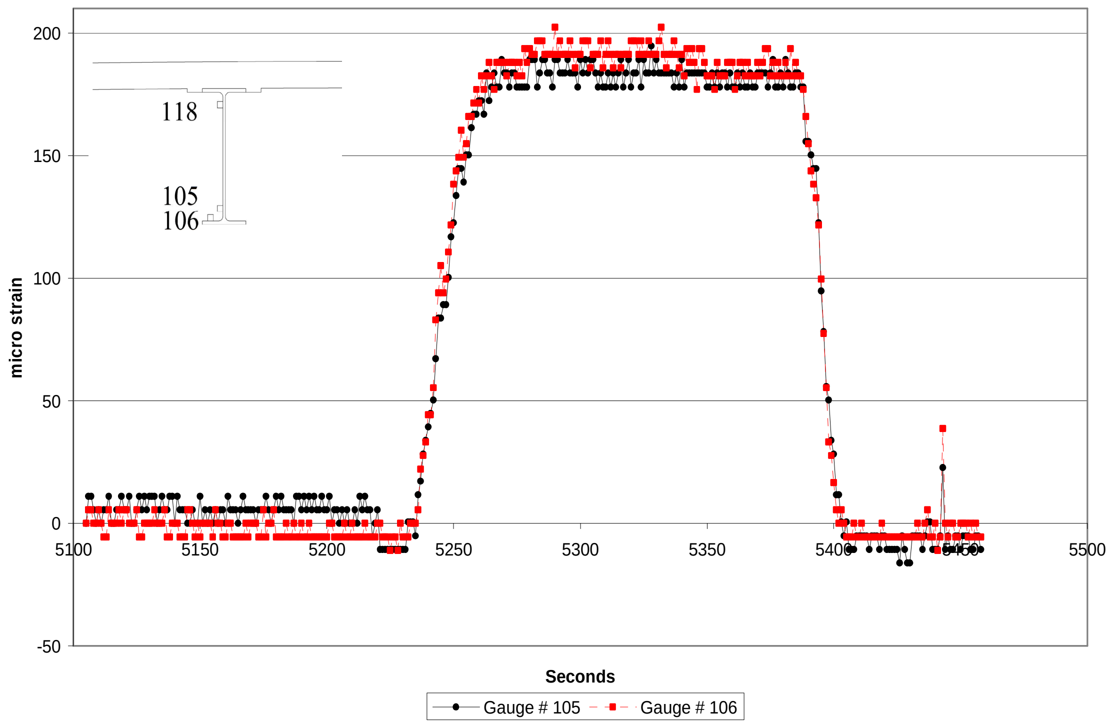

Figure 6.18 Girder Strains in Span #1 – Girder #3

Table 6.7 Deflection Results For Span #1

4Trucks	Potentiometers			
Test #	G1	G2	G3	G4
1	0.260	0.342	0.349	0.260
2	0.246	0.339	0.352	0.264
3	0.255	0.348	0.357	0.268
4	0.263	0.357	0.358	0.258
Measurement Average	**0.256**	**0.347**	**0.354**	**0.263**
FE Analysis	**0.293**	**0.410**	**0.429**	**0.328**

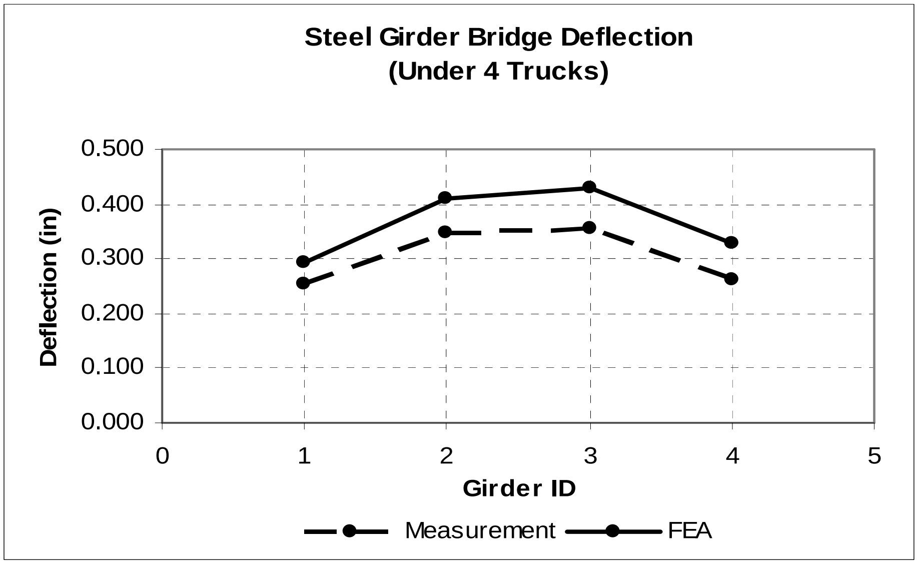

Figure 6.19 Deflection of Span #1

6.5.2.3 Pier Cap Strains

The results of the load tests to measure strains in the pier cap for bent 2 were inconclusive, as the strain readings from most sensors did not exceed the noise level of the instrumentation. The LVDTs have an error of 0.1% which in the case of the sensor configuration in the test equates to +/- 10με (microstrain). As indicated in Tables 6.8, 6.9, and 6.10, the strains in most of the sensors did not exceed this margin of error. However, the FE analysis also indicated that the diagonal tension strains in the pier cap would not exceed the tensile splitting strain in the concrete under test conditions. In particular, when the dead load on the structure is taken into account, the additional tensile strain from the test vehicles required to cause cracking is approximately 50με. In this respect, the measurements, FE analysis, and inspection of the pier cap all are consistent, in that all indicated that the strain did not exceed the cracking strain. Based on the FE analysis the bridge was already subjected to an estimated 90με caused by the dead load meaning a live load causing more than 60με would create cracking. Furthermore, no cracks were observed in the pier cap during or after the test. The only readings that exceed the margin of error of the sensors are those of gauges 121, 103, and 117 (see Figure 6.11 and 6.12), which were all mounted in the compression strut zone, as well as gauges 110 and 109, which both were mounted in the tension tie over the two columns. Gauges 125 and 112 were mounted in compression struts created by the outer girders; however the loads were from the outer girders were much smaller than the inner girders, and as a result these gauges did not record significant compression strains.

6.5.3 Discussion and Conclusions

The span 1 deflection measurements are in good agreement with the FE analysis results, as with the three previous bridge tests, and show that a properly developed FE analysis is an effective means of modeling slab/girder behavior. In the case of this particular bridge, it is believed that these results could be improved with more definitive knowledge of the degree of composite action in the slab and girders and the behavior of the girders at the abutment supports. Unfortunately the LVDTs designed to measure strain in the girders did not identify the degree of composite action due to an inoperative gauge. Moreover, the measurements from these gauges did not agree as well with the FE results as the deflections. It seems clear that more reliable verification of overall structural system behavior can be gathered from measurements of structural displacements than measurements of local strains.

Table 6.8 LVDT Micro Strain Readings (+/-10με) in Gauges on West Face of Pier Cap (- Compression)

West Face of Pier Cap

	Rosette 1 (Opposite Rosette 4)			Rosette 2 (Opposite Rosette 5)			Rosette 3		
Gauge Location		C Strut	T Strut		C Strut	T Strut		C Strut	T Strut
Gauge #	123	125	124	122	121	126	114	103	104
Test 1	2.0	1.0	2.0	-5.9	-27.8	-3.8	5.2	-23.5	-2.2
Test 2	-4.3	-10.9	3.9	-8.4	-24.8	-0.4	-0.8	-20.9	3.8
Average	**-1.1**	**-5.0**	**3.0**	**-7.1**	**-26.3**	**-2.1**	**2.2**	**-22.2**	**0.8**

Table 6.9 LVDT Micro Strain Readings (+/-10 με) in Gauges on East Face of Pier Cap (- Compression)

East Face of Pier Cap

	Rosette 4 (Opposite Rosette 1)			Rosette 5 (Opposite Rosette 2)		
Gauge Location		C Strut	T Strut		C Strut	T Strut
Gauge #	111	112	113	115	117	116
Test 1	1.9	-11.9	0.8	-0.4	-21.0	4.2
Test 2	2.1	-12.1	-0.5	1.9	-18.5	7.1
Average	**2.0**	**-12.0**	**0.1**	**0.8**	**-19.8**	**5.6**

Table 6.10LVDT Micro Strain Readings (+/-10με) in Gauges on Top of Pier Cap (- Compression)

Gauge #	110	109
Test 1	25.3	14.3
Test 2	37.6	27.7
Average	**31.4**	**21.0**

While the results of the pier cap test support the notion that its behavior in shear is best modeled by a strut and tie analysis, they fall short of demonstrating this conclusively due to the close proximity of many of the results to the sensor's error range. Further study of this matter is needed to come to a definitive conclusion regarding the pier cap behavior in shear. During the course of the load test the bridge was subjected to a total load of 259,400 lbs (1,154 kN); each of the four trucks used in this test exceeded the GDOT posted limit for this bridge by 18,850 lbs (84 kN). According to the analysis performed by the GDOT using the LF rating method, the bridge was severely overloaded and yet no damage was observed during the load tests. On the other hand, using the strut and tie analysis presented in the AASHTO's LRFD Specification as the basis for calculating the shear capacity, the trucks used in this test did not overload this bridge. This finding lends further support to the use of strut and tie analysis to model the shear capacity of deep beam sections such as pier caps.

CHAPTER 7 CONCLUSIONS

This report has described the load testing and detailed analysis phase of an in-depth study to examine current bridge rating procedures in the State of Georgia using an integrated set of advanced analytical and experimental techniques. The economic impact of posting or closing a bridge unnecessarily is substantial; at the same time, public safety is endangered by not taking appropriate action to maintain, rehabilitate or replace a bridge that is structurally deficient. The fact that roughly 2,000 of the approximately 9,000 bridges under the jurisdiction of the GDOT have been found to require posting by current rating methods warrants this examination of current rating practices.

Four bridge structures that are representative of non-interstate girder bridges that are of concern to GDOT's bridge engineering and maintenance staff were selected for detailed analysis. These bridges included two reinforced concrete T-beam bridges, one with a straight approach and the second with a skewed approach, one steel I-girder bridge, and one prestressed I-girder bridge. The straight reinforced concrete and steel girder bridges are older structures designed for H-15 loads (Figure 2.2); the current legal H type truck weighs 23 tons, 8 tons more than the load for which these structures were designed. The skewed bridge, while designed for the current HS-20 design load, was selected because current rating procedures have caused it to be posted. The prestressed girder bridge also was designed for current HS-20 loads; however, it was selected due to questions raised by GDOT bridge maintenance personnel regarding performance of prestressed bridges designed in the 1980's.

The current GDOT rating practices utilize mainly the LF method, as defined in AASHTO's *Manual for Condition Evaluation of Bridges, Second Edition* (1994). Independent ratings of each of the four bridges selected for further investigation in this research program were performed using the same LF method as that used by the State; ratings also were performed using the AASHTO allowable stress (AS) and load and resistance factor rating (LRFR) methods. The ratings from the AS and LF methods were compared and found to be approximately 20% higher than the LRFR ratings for the reinforced concrete bridges and the prestressed concrete bridge. The most notable difference, however, was in the ratings for the steel girder bridge (Chapter 6), where the rating of the pier cap supporting the steel girders determined the rating (and posted loads) for the bridge. Pier caps with the particular configuration found in this bridge have short shear spans and behave as deep beams; the load-carrying mechanism in such beams is better modeled by the strut and tie method than by the traditional ACI 318 model. The use of this new capacity calculation method is permitted by the LRFR option in the AASHTO *Manual for Bridge Evaluation, First Edition (2008).* The rated capacity of the pier cap increases by up to 59% in the steel girder bridge examined when the compression field model is used, but preliminary investigations of similar bridge pier caps indicate that the level of conservatism is dependent on the dimensions of individual pier caps and the placement of the girders that they support.

Finite element models of the four bridge structures were developed using ABAQUS, a commercially available finite element platform capable of modeling nonlinear material behavior (Appendix A). These finite element models were used to perform a preliminary assessment of bridge performance during diagnostic load testing and to assist in the design of the actual load tests, which were conducted between September, 2006 and May, 2007. The bridge test results, in turn, were used to validate and improve the finite element modeling process for typical bridges in Georgia. It was determined that the best overall assessment of bridge behavior and load-sharing among the girders would be gained from measurements of girder deflections in each of the four bridges. To this end, the Georgia Tech team instrumented each of the four bridges using potentiometers that were monitored in real-time during the test using a data acquisition system. Dial gauges were also installed and monitored as an independent check on the potentiometer measurements. In addition, the behavior in shear of the pier cap supporting the steel girder bridge was of interest, as noted above, and was instrumented to measure strain using LVDT's.

The deflection measurements in all cases were in good agreement with those predicted by the FE model. With the exception of the prestressed girder bridge test, all other bridges were loaded well above their design or posting limits; yet, the responses of all four bridges remained well within the elastic limit when loaded to the maximum intensity with four trucks. The maximum deflections measured during the load tests were on the order of 25% - 50% of the span/800 limit on deflection stipulated in the AASHTO design specifications.

One of the specific objectives of the load testing was to gage the accuracy of the girder distribution factors recommended in the AASHTO design and rating guidelines. To accomplish this the measured deflections were used to validate the finite element model of each bridge. The models were subsequently used to develop girder distribution factors comparable to those used in AASHTO. Table 7.1 compares the various AASHTO girder distribution factors to those computed in this study.

Table 7.1: Comparison of the moment distribution factors for interior girders

Bridge Type	**LFR/ASR**	**LRFR**	**FEM**
Concrete T	0.597	0.69	0.407
Concrete Skew T	0.757	0.73	0.482
Prestressed	0.818	0.85	0.521
Steel Girder	0.725	0.72	0.513

The strain measurements in the pier cap of the steel girder bridge unfortunately were too close to the noise range of the LVDTs to determine conclusively how well they agreed with the FE model predictions. The strain readings were low in both the FE analysis and actual structure under the applied loading, well below the cracking strain of the concrete (estimated at approximately 150 microstrain) in diagonal tension. Additionally, no cracking was observed in the pier cap during testing, despite the fact that each of the four trucks used to load the structure was an average of 18,850 lbs (84 kN) over the posted limit for the bridge determined by the GDOT using the LFR rating method and as outlined by AASHTO in the 1994 *Manual for Condition Evaluation*. In contrast, the independent analysis using the strut and tie method proposed by AASHTO in the 2003 *Manual for Condition Evaluation* determined that the trucks used during the test did not exceed the capacity of the pier cap. Additionally the LRFR based analysis of the pier cap showed that instead of posting the bridge for a 25 ton HS-20 truck it could be posted for a 34 ton HS-20 truck.

The experience with these four load tests demonstrates that the best approach to assessing a bridge's overall performance is through measures of global response such as displacement. Basing an assessment of *in situ* performance on global response measurements, as opposed to local responses such as strain, minimizes questions and concerns about the significance (if any) of local non-homogeneous or material behavior. It was also observed that redundancy in measurements, through multiple gauges at a single location and gauges at multiple locations in a single element, should be utilized whenever practical.

This experimental and analytical investigation has shown that properly developed FE bridge models can represent the behavior of common types of bridge structures through a broad range of load intensities. The development of improved guidelines for rating existing bridges will rely heavily on these validated finite element models of typical bridges. Task 4 of the overall research effort will develop specific practical recommendations to improve the process used for rating bridges in Georgia. This future work will include a more detailed examination of the differences between LFR and LRFR rating methods, an investigation of the proper role of the strut and tie method in rating pier caps and other bridge components that behave as deep beams, development of practical guidelines for performing rating calculations, and recommendations for future load testing of bridges

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REFERENCES

American Association of State Highway and Transportation Officials (2007). *LRFD Bridge Design Specifications, 4th Edition* (including 2008 and 2009 interim revisions). AASHTO, Washington D.C.

American Association of State Highway and Transportation Officials (1994). *Manual for Condition Evaluation of Bridges, 2nd Edition* (including 1995, 1996, 1998 and 2000 interim revisions). Washington, DC: AASHTO, 2000.

American Association of State Highway and Transportation Officials (2003). *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges, 1st Edition* (including 2005 interim revisions). AASHTO, Washington D.C.

ABAQUS/standard User's Manual-Version 6.6 (2006). ABAQUS, Inc., Pawtucket, R.I.

Crisfield, M. A.(1986). "Snap-Through and Snap-Back Response in Concrete Structures and the Dangers of Under-Integration." *International Journal for Numerical Methods in Engineering* 22:751–767

Hillerborg, A., M. Modeer, and P. E. Petersson (1976). "Analysis of Crack Formation and Crack Growth in Concrete by Means of Fracture Mechanics and Finite Elements." *Cement and Concrete Research* 6: 773–782.

Jaramilla, Becky and Huo, Sharon. "Looking to Load and Resistance Factor Rating." Public Roads Magazine July/August 2005. Vol 69 No 1 <http://www.tfhrc.gov/pubrds/05jul/09.htm>.

Kupfer, H. B., and K. H. Gerstle (1973). "Behavior of Concrete under Biaxial Stresses." *Journal of Engineering Mechanics Division*, ASCE 99: 853-864.

O'Malley. C., N. Wang, B. R. Ellingwood and A.-H. Zureick (2009). Condition assessment of existing bridge structures: Report of Tasks 2 and 3 - Bridge Load Testing Program. GDOT Project Report RP05-01. (ftp://ftp.dot.state.ga.us/DOTFTP/Anonymous-Public/Research_Projects/).

Todeschini, C., A. Bianchini, and C. Kesler (1964). "Behavior of concrete columns reinforced with high strength steel." *ACI Materials Journal* 61(6): 701-716

Wang, N., Ellingwood, B.R., Zureick, A.-H. and O'Malley, C. (2009). Condition assessment of existing bridge structures: Report of Task 1 – Appraisal of state-of-the-art of bridge condition assessment. GDOT Project RP05-01. (ftp://ftp.dot.state.ga.us/DOTFTP/Anonymous-Public/Research_Projects/)

Appendix – A

Finite Element Modeling of Bridges

Improved bridge rating guidelines require the development and validation of finite element models (FEM) to predict the behavior of candidate bridges subjected to extreme service load events. Once validated by diagnostic load tests of four typical bridges, as described in this report, these FEMs can then be applied to conduct "virtual load tests" to predict the load capacity of other bridges of interest in the Georgia Bridge Inventory and to serve as a basis for developing improved and practical bridge rating guidelines.

Three-dimensional (3D) nonlinear FEMs of the superstructure of each sample bridge were constructed using ABAQUS (ABAQUS, 2006) before diagnostic load tests were conducted. All FEMs were developed according to bridge design and construction documents secured from the Georgia Department of Transportation. The FE analyses were performed using anticipated vehicle weights and arrangements to assist in designing the test instrumentation, to identify test vehicle locations, and to anticipate and guard against potential bridge vulnerabilities that might become apparent during the diagnostic load tests. Prior to the conduct of the load tests, actual test vehicles were weighted, and during the test, their placement on the bridge spans was measured. Following the load tests, FE analyses were redone using the actual test vehicle/axle weights and truck placements on the bridges, and predicted responses were compared with test measurements to determine the accuracy with which FEA can predict bridge behavior. The focus of the comparisons is on predicted vs measured bridge girder deflections because deflections are most accurately measured and easily interpreted in the *in situ* tests of the candidate bridges. This Appendix presents the FE modeling process and summarizes the results of the FE analyses of the four bridges.

A.1. Bridge ID 129-0045 (RC T-beam, Gordon County)

The Gordon County Bridge is a straight-approach reinforced concrete T-beam bridge with eight simple spans, described in detail in Chapter 3. The eastern exterior span was chosen for testing and modeling, based on its ease of accessibility. The selected span is 40 ft (12.2 m) long, and is supported by four beams. Transverse concrete diaphragms are located at each end of the span. According to the design drawings, the parapets and railings are non-composite with the bridge deck and therefore are not considered in modeling. However, their weights are included in the overall load analysis.

The concrete deck, girders and transverse diaphragms all are modeled using 3D continuum solid elements with a mesh of 3in x 3in x 3in (76 mm x 76 mm x 76 mm). The failure mechanism in the concrete is assumed to be either cracking in tension or crushing in compression. The stress-strain curve proposed by Todeschini (1964) is utilized to model concrete behavior under compression; in tension, the stiffness and strength reductions caused by cracking are taken into account by the smeared crack technique (Kupfer, 1973, Hillerborg, 1976 and Crisfield, 1986), in which crack initiation is based on strength criteria and crack propagation is based on the energy criteria of fracture mechanics. The compression strength of the concrete in the Gordon County Bridge is 2500 psi (17.25 MPa).

The steel reinforcement in flexure is modeled using a distributed approach, in which the reinforcing bars are smeared into membrane layers and embedded in the concrete at appropriate locations. The cross-sectional area of each layer is equal to the total cross-sectional area of the steel bars at the corresponding location. Each layer is governed by a uniaxial elastic-plastic stress-strain relationship and is modeled by 4-node quadrilateral plain stress elements. The compatibilities between reinforcing layers and concrete bricks are enforced by constraining the translational degrees of freedom of the embedded nodes (reinforcement layers) to the interpolated values of the corresponding degrees of freedom of the host elements (concrete bricks). Shear reinforcement is ignored in the FE model of the bridge superstructure. Based on the bridge construction documents, the yield strength of the reinforcing bars is assumed to be 40 ksi (276 MPa). The span is simply supported by the pier cap and abutment; these supports are modeled by a pin-roller boundary condition. Figure A.1 illustrates the FE model of the span of the Gordon County Bridge that was load-tested; the FEM has 420,928 degrees of freedom[1]. The FE modeling of the remaining three bridges described subsequently is at a similar level of resolution.

The FE analysis predicts that when all four trucks are positioned according to the test arrangement illustrated in Figure A.2, the maximum deflections in girders 1, 2, 3, and 4 are 0.18, 0.26, 0.27, and 0.19 inches (4.6, 6.6, 6.9, and 4.8 mm), respectively, which is consistent with the deflections in these girders of 0.15, 0.27, 0.27, and 0.20 inches (3.8, 6.9, 6.9 and 5.1, mm) that were measured by potentiometers during the load test. The maximum error as indicated in Figure A.3 is about 20%. This agreement is considered acceptable, particularly in light of the uncertainties regarding *in situ* construction that are inevitably involved in such a comparison.

A.2. Bridge ID 015-0108 (RC T-beam, skewed approach, Bartow County)

The Bartow County Bridge is a 12-span concrete T-beam bridge, in which the centerline of the bridge roadway is skewed 30 degrees with respect to its supports. The bridge is described in detail in Chapter 4. The end span selected for testing is 40 ft (12.2 m) in length, with five beams that are simply supported by the pier cap and abutment. Because of the similarity of this bridge to the Gordon County Bridge, the FE modeling process for this span is virtually identical to the process described previously for that bridge.

The truck arrangement during the load test with all four trucks in place is illustrated in Figure A.4. The maximum deflections predicted by the FE analysis under

[1] The FE models in this study, once validated, are intended to be used to conduct "virtual load tests" of arbitrary bridges in the State of Georgia inventory in support of improved rating guidelines. The resolution of these FE models must be sufficient for this purpose. There is no implication that FE modeling at this scale of resolution is necessary or desirable for bridge design or bridge rating in general.

test condition are 0.041, 0.128, 0.170, 0.130, and 0.040 inches (1.04, 3.25, 4.32, 3.30, and 1.02 mm) in girders 1 – 5, respectively. For comparison, these predicted deflections together with the girder deflections measured by potentiometers during the test are presented in Figure A.5. In the instance of Girder 3, the measured deflection is 0.153 inches (3.89 mm), a difference of 11%. As with the previous bridge, the FE analysis results and the load test measurements indicate good agreement.

A.3. Bridge ID 223-0034 (Pre-stressed Concrete, Paulding County)

The test span for the Paulding County Bridge is 68 ft (20.7 m) with five AASHTO Type III girders supporting the bridge deck. The test span is simply supported. The construction documents show that the reinforced concrete deck and pre-stressed concrete girders were designed for composite action under live load, and the parapets and railings were constructed as non-composite with the deck. Three transverse concrete diaphragms are located at the centerline and two ends of the span, respectively. This bridge is described in further detail in Chapter 5.

The concrete portion of the structure is modeled by 3D continuum solid elements, while the steel reinforcement in the bridge deck and transverse diaphragms is modeled by layers embedded in the solid concrete elements at the appropriate positions, as described previously. The pre-stressing strands in the girders are modeled individually using truss elements with the same cross-sectional properties as the strands; these truss elements then are embedded in the solid concrete elements by restraining the translational degrees of freedom of the truss element nodes to the interpolated values of the corresponding degrees of freedom of the adjacent concrete solid element nodes. The effect of pre-stressing is simulated by equivalent balanced forces: horizontal compression forces applied at each end of the girders and distributed uplift forces caused by the drape in the strand profile at the center of the girders. The pre-stress in strands is replicated by applying to the truss elements an initial stress condition, with the magnitude of effective pre-stress indicated in the design documents.

The composite action between the girders and the bridge deck is modeled by multiple point constraints (MPC), which provide a rigid link between two nodes to constrain all degrees of freedom at slab node to the corresponding degrees of freedom at the adjacent supporting beam node. The simple supports of this span are modeled by pin-roller boundary conditions. The constitutive model for the concrete is defined by the Todeschini (1964) stress-strain relationship in the compression zone and by the smeared crack approach on the tension side. Steel bars and pre-stressing strands both are assumed to be elastic-plastic in their stress-strain behavior.

The location of the trucks during the Paulding County Bridge test is shown in Figure A.6 and the FE analysis results and test measurements are compared in Figure A.7[2]. With four trucks on the span, the FE analysis predicts the maximum deflection in girders 1, 2, 3, and 4 to be 0.091, 0.171, 0.215, and 0.181 inches (2.3, 4.3, 5.5, and 4.6

[2] As noted in Chapter 5, the potentiometer installed on Girder 5 malfunctioned during the load test.

mm), respectively, which differs by an average of 15 % from the test measurements as indicated in Figure A.7.

A.4. Bridge ID 085-0018 (Steel Girder, Dawson County)

As described in Chapter 6, the Dawson County Bridge contains four simply supported spans. The exterior span selected for testing is 49 ft (14.9 m) in length, with reinforced concrete deck supported by two W33 x 118 exterior girders and two W33 x 130 interior girders. Concrete transverse diaphragms are located at the centerline and two ends of the span, respectively. According to design drawings, the steel girders and reinforced concrete deck were designed for non-composite action, and the parapets and railings were designed as non-composite with the deck as well.

Each of the steel girders is modeled using beam elements for the flanges and shell elements for the web, with elastic-plastic stress-strain material properties. The concrete deck and diaphragms are modeled using the same elements and material definitions that were used for the decks and transverse diaphragms described previously for the other bridges. The composite action between the steel girders and the concrete deck is realized using MPC, as described in the last section. The span is pinned to the concrete pier cap at one end and rests on the abutment on the other end, so the supports are modeled with pin-roller boundary conditions.

A comparison of the FE analysis results and field measurements under the test load condition involving four trucks (Figure A.8) is presented in Figure A.9. The maximum deflections in girders 1 – 4 predicted by the FE analysis are 0.293, 0.410, 0.429, and 0.328 inches (7.4, 10.4, 10.9, and 8.3 mm), respectively. The deflections measured by the potentiometers for the same girders are 0.256, 0.347, 0.354, and 0.263 inches (6.5, 8.8, 9.0, and 6.7 mm), respectively. The average error is about 20%. Although this agreement is not unreasonable, the fact that the support conditions at the abutment or the degree of composite action could not be verified in this bridge may explain the difference between the predicted and observed deflections.

A.5 Conclusion

The comparison between FE analyses and load test results indicates good agreement. The discrepancies between predicted and observed deflections invariably are within about 20%, as previously discussed; in the majority of cases, the differences are substantially less. Such differences can be attributed to various uncertainties associated with experimental data collection under field conditions and the many assumptions made in the FE analyses, including homogeneity and magnitude of *in situ* material properties, and idealized boundary conditions. In view of these factors, results of the FE analyses of the four test bridges are considered sufficient to describe and quantify the load-bearing mechanisms that affect the bridge capacity and its load ratings.

These preliminary analyses of typical bridges in the State of Georgia have revealed that the load ratings calculated by FE models that have been validated by diagnostic tests substantially exceed the load ratings that are obtained by the "Load Factor Rating (LFR)" method (AASHTO, 1994) that currently is used by the State of Georgia for this purpose. For the Bartow County Bridge, for instance, the LFR procedure results in rating factors of 1.30 for the HS-20 design load, while the rating by FE analysis of this bridge for the same vehicle is 1.86, presenting an increase of 43%. The actual distribution of load to the supporting girders is a major factor contributing to this difference. The data comparison shown in Figure A.5 indicates that only about 33% of the test load actually went into the middle girder of the Bartow County Bridge, while the girder distribution factor in the current LFR method would require that 70% of the test load be apportioned to that girder. Other factors which are revealed by the 3D FE analysis but have been ignored by rating guidelines, such as additional stiffness from transverse diaphragms and actual supports conditions, can also contribute significantly to the difference between load ratings by FE methods and those calculated according to the LFR guidelines.

Accordingly, this project utilizes FE modeling, as validated by the diagnostic load tests of these statistically representative sample bridges, to characterize the likely performance of a broad selection of bridges in the Georgia bridge inventory. This work makes it possible to integrate the bridge-specific heuristics utilized by experienced bridge engineers with the advanced structural analysis tools to provide reliable and measurement-based determination of the bridge serviceability and load capacity. Further research findings are presented in the Task 4 report.[3]

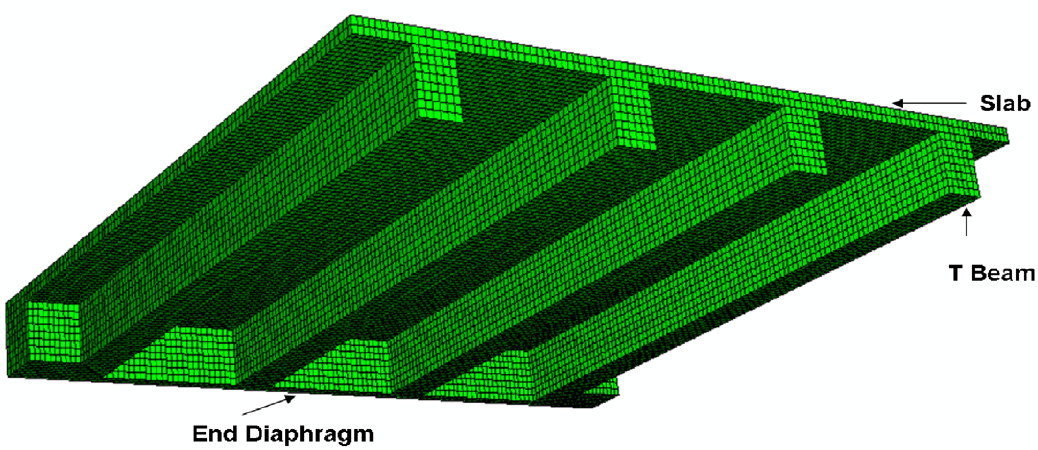

Figure A.1: A Snapshot of the FEM of Gordon County Bridge

[3] Ellingwood, B.R., Zureick, A.-H., Wang, N. and O'Malley, C. (2009). "Condition assessment of existing bridge structures: Report of Task 4, Part I – Development of guidelines for condition assessment, evaluation and rating of bridges in Georgia." Report of Project GDOT No. RP05-01, Georgia Department of Transportation, Atlanta, GA (ftp://ftp.dot.state.ga.us/DOTFTP/Anonymous-Public /Research_Projects/).

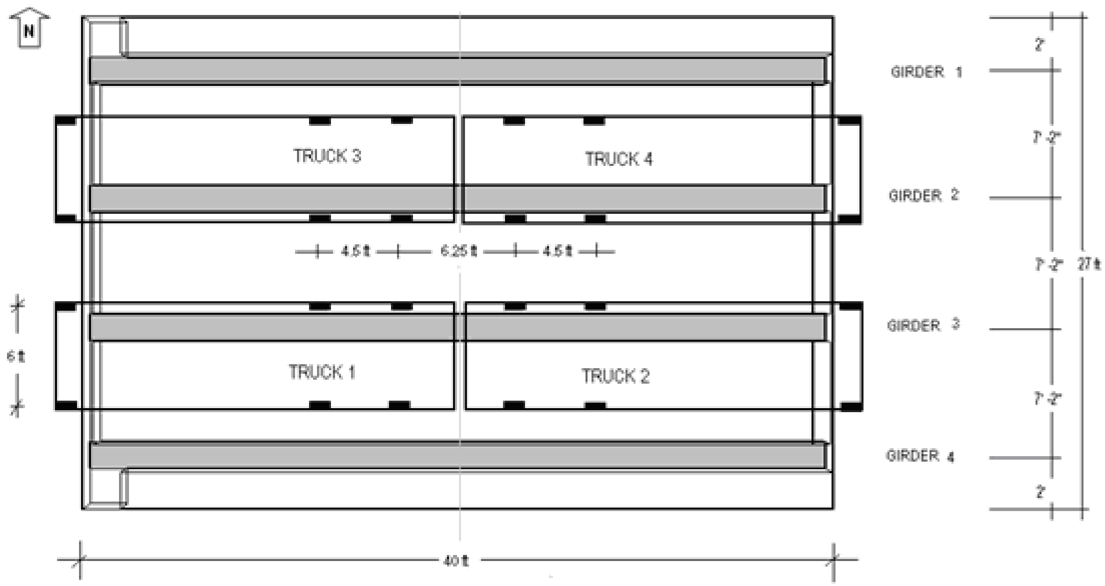

Figure A.2: Placement of Trucks for Gordon County Bridge Test

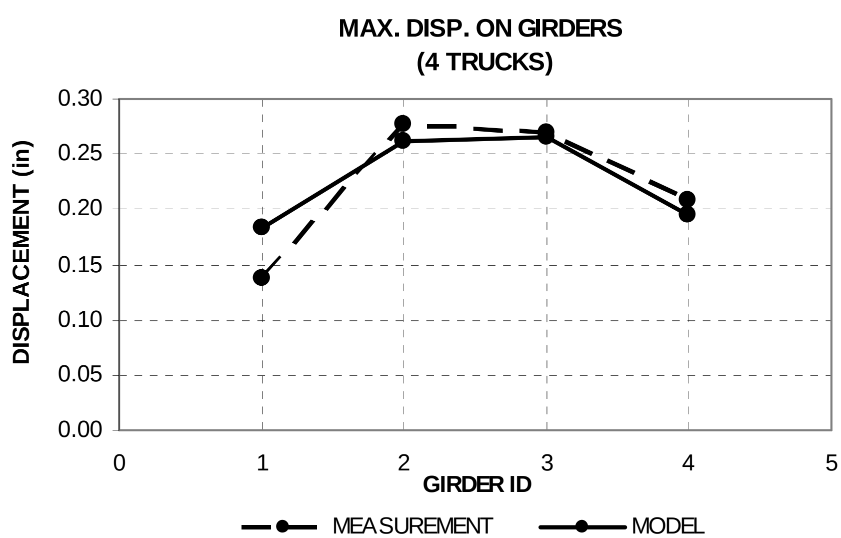

Figure A.3: Test measurements and FE analysis results for Gordon County Bridge

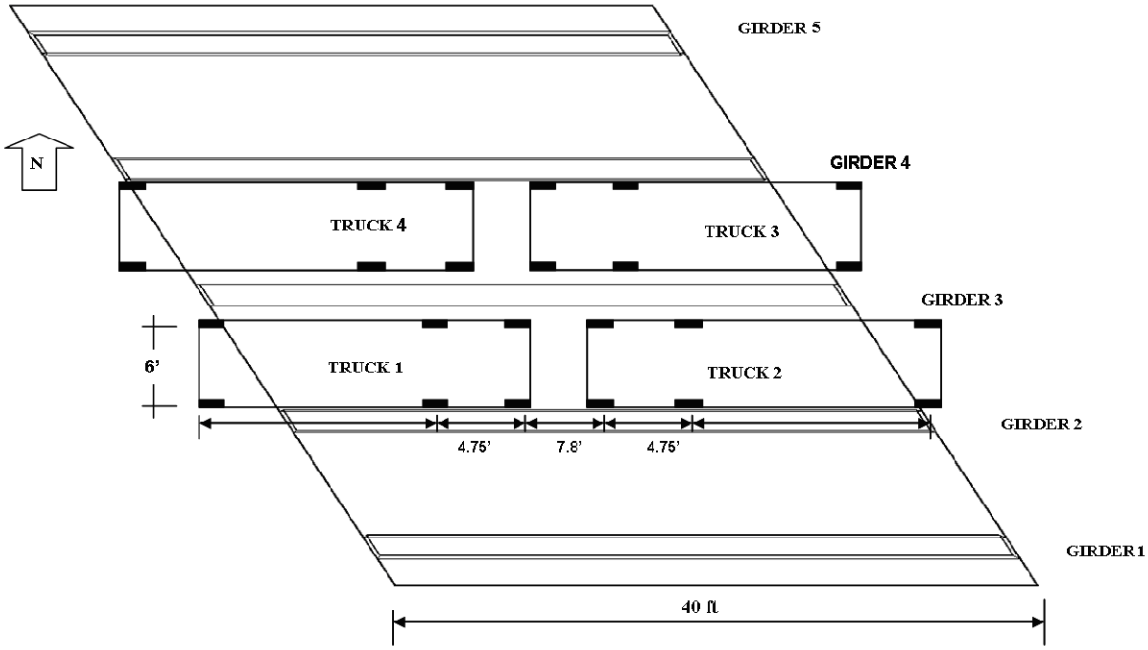

Figure A.4: Placement of Trucks for Bartow County Bridge Load Test

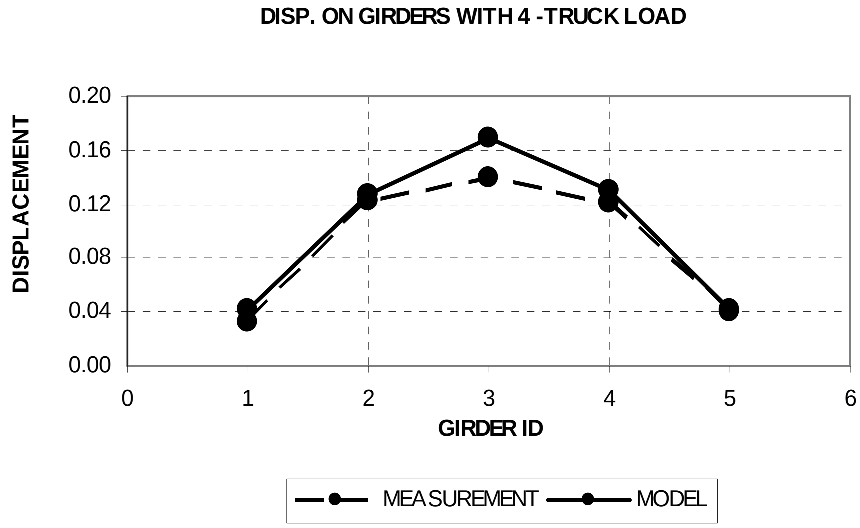

Figure A.5: Test measurements and FE analysis results for Bartow County Bridge

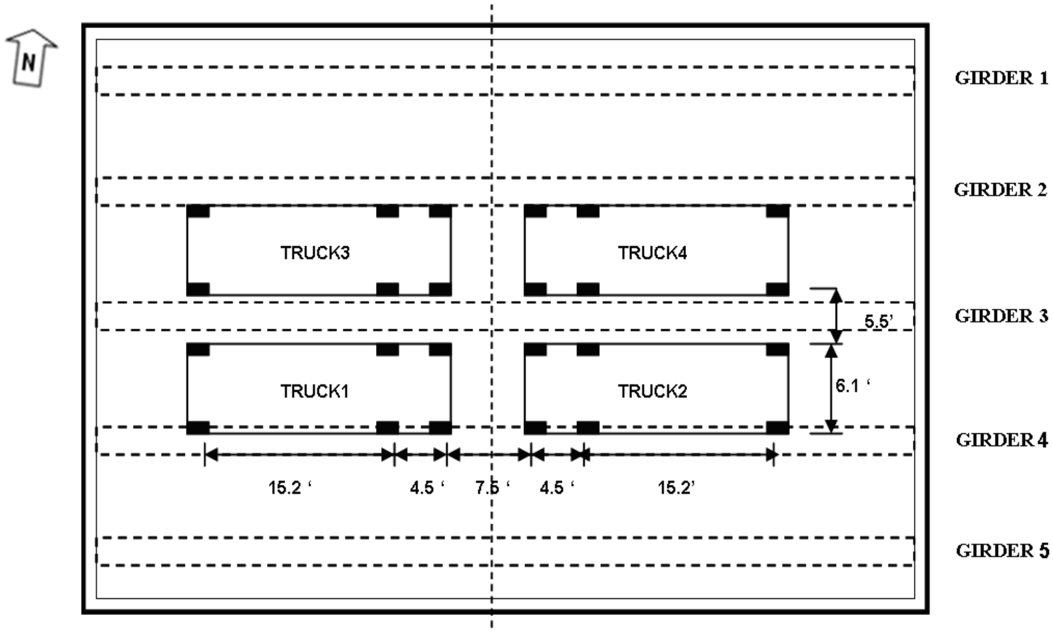

Figure A.6: Placement of Trucks for Paulding County Bridge Load Test

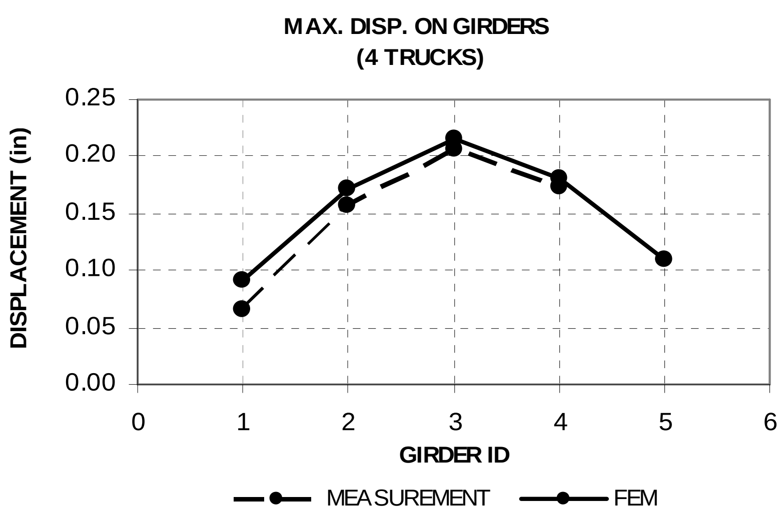

Figure A.7: Test measurements and FE analysis results for Paulding County Bridge

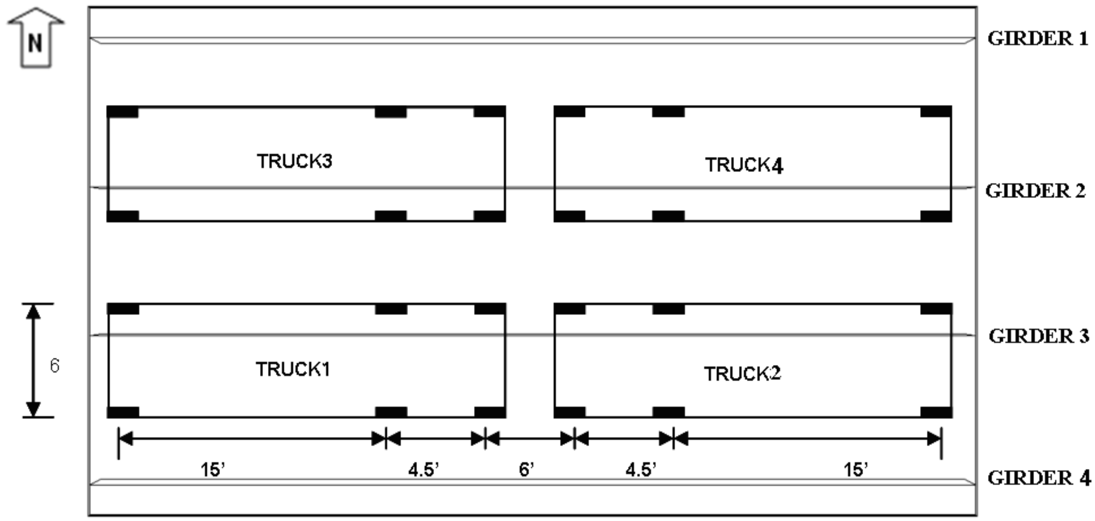

Figure A.8: Placement of Trucks for Dawson County Bridge Load Test

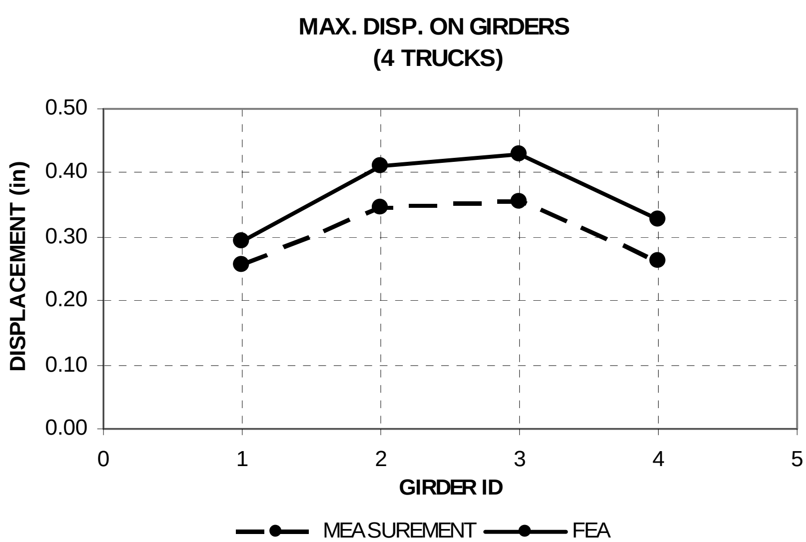

Figure A.9: Test measurements and FE analysis results for Dawson County Bridge